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Stability of streambanks subjected to highly variable streamflows: the Osage River Downstream of Bagnell Dam

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STABILITY OF STREAMBANKS SUBJECTED TO HIGHLY VARIABLE STREAMFLOWS: THE OSAGE RIVER DOWNSTREAM OF BAGNELL DAM

by

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ABSTRACT

Streambank erosion of the Osage River downstream of Bagnell Dam is naturally-occurring; however, it may be significantly worsened due to releases made from the dam to generate hydropower. In this study, six typical outflow release patterns from Bagnell Dam were evaluated to determine their effects, if any, on the stability and the rate and amount of erosion of the banks of the Osage River.

The Bank Stability and Toe Erosion Model (BSTEM), version 5.2, was used to carry out the erosion and stability calculations. The model was validated by using data from another study and comparing the results from BSTEM with those of the other study. BSTEM produced very similar results, and it was thus concluded that it would provide a reliable analysis for this study. The six outflow scenarios were evaluated, and those that resulted in the greatest amount of erosion and bank instability were identified based on the model results.

A sensitivity analysis was also completed to determine the input data necessary for BSTEM that would have the greatest impact on the model outcome. The input parameters evaluated in the sensitivity analysis included various geotechnical properties, assumptions regarding depth to the phreatic surface, and the timing and slope of the outflow hydrographs.

The results of this study indicate that the streambanks of the Osage River are quite stable when erosion of the noncohesive toe material is not considered; however, when erosion is accounted for, most of the banks become unstable during the outflow scenarios. Based on the typical bank stratigraphy, the most common failure mechanism that would be encountered on the Osage River is mass wasting or cantilever failures.

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1. INTRODUCTION

1.1. GENERAL

Bagnell Dam is located in Camden County, Missouri, and impounds water from the Osage River, forming Lake of the Ozarks. Construction of Bagnell Dam began in August of 1929 and was completed in April of 1931. The dam is a concrete gravity dam measuring 2,543 feet in length, with a spillway controlled by five tainter gates. The total length of the spillway is 520 feet [1].

The dam, owned by Ameren UE, was constructed for the purpose of generating hydroelectric power. Downstream of the dam, the Osage River flows for 80 miles to its confluence with the Missouri River. The demand for hydroelectric power generation varies greatly depending on the time of year, the time of day, as well as other factors. Because of this, there are generally large fluctuations in the discharges from Bagnell Dam that occur within a relatively short period of time. Erosion and subsequent streambank instabilities are naturally-occurring processes in rivers such as the Osage River; however, they can be significantly exacerbated due to human activities, including releases made from dams to generate hydroelectric power [2].

The purpose of this thesis is to quantify the potential effects of various hydropower release scenarios from Bagnell Dam on erosion and bank instability for several cross-sections on the Osage River downstream of the dam. This was accomplished by calculating the amount of toe erosion that occurred at various times throughout each scenario, as well as the corresponding Factor of Safety (FS). The eroded cross-sections were also plotted alongside the original cross-section geometry to demonstrate visually the changes that occurred due to erosion. Both the toe erosion and the FS were determined using the program BSTEM (Bank Stability and Toe Erosion Model), created by the National Sedimentation Laboratory, Agricultural Research Service (U.S. Department of Agriculture) [3]. BSTEM was created specifically to investigate the erosion and stability of streambanks.

1.2. FLOW RELEASE SCENARIOS

Operating records covering a period from May 2001 through October 2001were provided by Ameren UE for a previous erosion analysis completed in 2003 [2]. The overall outflow hydrograph from the dam during this time period was divided into six outflow scenarios that represent the most common release patterns from the dam. Each of these scenarios is described in detail below:

- **Lake Destratified and Oxic Hypolimnia Period ("Hypolimnia")**
  The portion of the hydrograph representing this outflow scenario occurred between May 24 and June 11, and exemplifies the releases typically made from the dam in early summer when temperatures are rising, increasing the demand for hydroelectric power. This scenario will also be referred to as outflow scenario 1.
- **Extended High Flow Generation Period ("High Flow")**
  The releases made during the time period from June 11 through July 7 are considered to be representative of typical releases of the High Flow period. High Flow outflows are not dependent on the time of year, rather they are caused by heavy rainfall within the Bagnell Dam drainage area and/or by flood releases from Harry S. Truman Dam, which is located upstream. The duration of this type of outflow scenario varies considerably, potentially lasting anywhere between one day and several weeks. This scenario will also be referred to as outflow scenario 2.
- **Typical Summer Generation Period ("Summer Generation")**
  The portion of the hydrograph between July 7 and August 9 illustrates normal Summer Generation releases. During this period, electricity demands are large due to the high temperatures of summer, and considerable peaks in the demands occur throughout the day. Typically the peak electricity demands occur early in the morning, in the late afternoon and in the early evening. The size and timing of the outflows are dependent on not only electricity demand, but also on the amount of water available in the reservoir for release. The outflows range anywhere from the continuous minimum of 450 cubic feet per second (cfs) to 34,000 cfs. Normally, a release made during a peak demand time consists of an abrupt ramp up to meet the electricity need, followed by a period of fairly continuous releases which then taper off once the need has been met. This scenario will also be referred to as outflow scenario 3.

- **Typical Summer Low Flow Period ("Summer Low Flow")**
  The releases made during the period from August 9 to August 25 are typical of Summer Low Flow releases. During this time period, rainfall and subsequent runoff into Lake of the Ozarks is fairly low; thus releases are restricted in order to minimize water withdrawals from the lake to maintain a specific level for recreational purposes. During most of this period the outflow from the dam will be the minimum required outflow of 450 cfs; however, it is possible to see sporadic, short-term peaking releases. This scenario will also be referred to as outflow scenario 4.
- **Late Summer Transition Period ("Late Summer")**
  The portion of the hydrograph between August 25 and September 18 demonstrates typical releases during the Late Summer period. During this time there is generally an increase in rainfall and subsequent runoff into Lake of the Ozarks; however, temperatures and electricity demand remain high. The increase in rainfall allows for more liberal releases to be made to meet the electricity demand while maintaining the lake level necessary for recreational purposes. This scenario will also be referred to as outflow scenario 5.
- **Fall Oxygen Rebound Period ("Fall Rebound")**
  During the fall months, temperatures tend to decrease slightly, lessening the electricity demand. At the same time, rainfall typically increases, providing adequate inflow to Lake of the Ozarks to maintain appropriate lake levels while allowing for peaking releases as necessary. The last portion of the hydrograph from September 18 through October 31 typifies the releases made during the Fall Rebound Period. This scenario will also be referred to as outflow scenario 6.

The entire outflow hydrograph was used as input in an unsteady hydraulic model of the Osage River reach between Bagnell Dam and the Missouri River.

1.3. PREVIOUS EROSION ANALYSIS

A previous analysis (hereafter referred to as the 2003 erosion analysis) of the effects of hydropower generation releases on erosion and bank stability downstream of Bagnell Dam was completed in 2003. The subsequent report which provides the results of this analysis is titled "Erosion Potential of the Osage River Downstream from Bagnell Dam" [2], and was one of the primary sources of data used for this study.

The 2003 erosion analysis quantified the effects of fluctuating discharges on the geometry and stability of the cross-sections using a program called Erosion Data Viewer (EDV). The EDV program determined the shear stress on the banks of the cross-sections, and the subsequent erosion rate for each soil type. The erosion rate for each time period was then multiplied by the duration of the time period to determine the total amount of erosion occurring during that time period. The total erosion potential (EP) for each bank over the course of each outflow scenario was calculated by summing all the erosion amounts for that scenario. The EPs for each scenario were compared in order to determine the effects of the various hydropower releases on the amount of erosion along the Osage River downstream from Bagnell Dam.

1.4. DATA AVAILABILITY

Station-elevation data for 13 cross-sections along the Osage River were available for use in this study. Cross-section geometry data was originally collected in 1997 for use in a HEC-2 hydraulic model of the river. This data was updated in 2003 to determine whether significant changes in cross-section geometry had occurred since the 1997 data was collected. The updated cross-section data was collected via bottom profiling and traditional surveying methods [2]. A map showing the location of the cross-sections used in the hydraulic model is provided in Appendix A, and the station-elevation data for each cross-section is provided in Appendix B.

Soil borings were drilled at the streambank of each of the 13 cross-section locations in order to determine the soil stratification to be used in the 2003 erosion analysis. The typical soil conditions found along the banks of the Osage River are fine-grained silts and clays along with fine to medium sands. Most of the bank material was normally-consolidated, and the most common bank stratigraphy encountered was a lower layer of loose sand and gravel with an upper layer comprised of silt and/or clay [2]. The soil distribution for each cross-section used for this erosion study is shown on the cross-sections provided in Appendix B.

Wells were installed at four of the 13 cross-section locations for the purpose of monitoring groundwater fluctuations within the banks. Although some data was collected at the observation wells, it was inadequate to provide detailed information regarding changes in the phreatic surface elevation as they relate to changes in the flow elevation in the channels; therefore the observation well data were omitted for the purposes of this study.

As previously discussed, an actual release hydrograph was provided by Ameren UE for the period from May 2001 to October 2001. The hydrograph provided outflow data from the dam every three hours.

Flow elevations at each of the cross-sections during the outflow hydrograph were also available, as they had been determined in a previous hydraulic model of the Osage River. The hydraulic model is discussed in more detail in the following section.

1.5. HYDRAULIC MODELS

The outflow hydrograph provided by Ameren UE is shown in Figure 1.1. The flows were originally used as input in a HEC-2 hydraulic model of the Osage River between Bagnell Dam and the Missouri River [4]. The HEC-2 model was developed by the US Army Corps of Engineers (USACOE) Kansas City District Office, and is a one-dimensional, steady state model that uses Manning's equation to calculate the water surface elevation at defined cross-sections along the river. The cross-section data for this model was collected in 1997 [4].

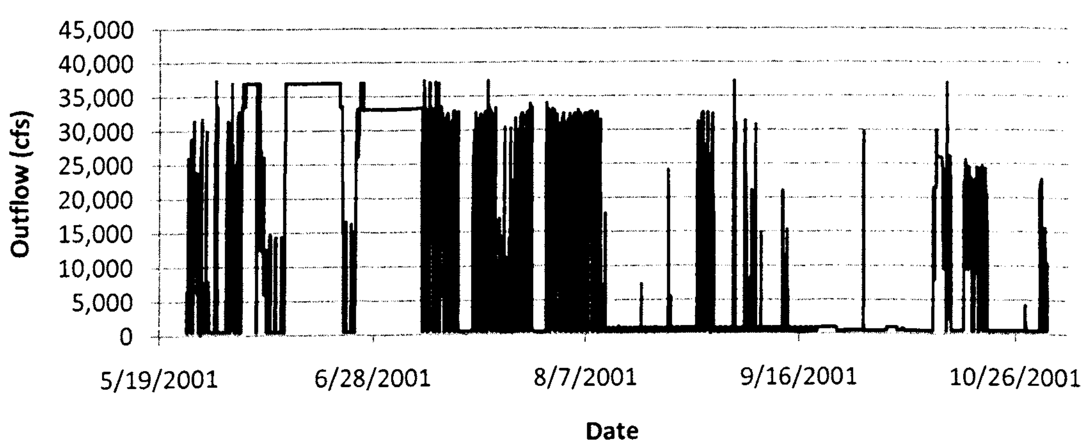

Figure 1.1 Typical Outflow Hydrograph from Bagnell Dam

In 2001, the original HEC-2 model was incorporated into an unsteady hydraulic model developed by Mead & Hunt (Madison, WI). Although the HEC-2 model provided the basis for the 2001 model, the cross-section geometry was updated and the model was calibrated using known flow elevation data for varying dam outflows. The downstream boundary condition for the model was a rating curve at the confluence of the Osage River and the Missouri River [5].

The flow elevations determined in the 2001 unsteady hydraulic model were used as input in the BSTEM model for this study. The amount of bank and toe erosion at each cross-section was calculated, followed by a calculation of the FS for the eroded profile at various points throughout the various outflow scenarios.

1.6. PURPOSE AND SCOPE

The purpose of this study is to investigate the effects of the flow releases from Bagnell Dam on the erosion and stability of the banks along the Osage River downstream from the dam. Furthermore, the validity of using BSTEM to determine bank stability will be evaluated, and in this way provide a basis for future use of the program.

The scope of this study involved a review of literature pertinent to bank stability analysis, collection and review of the available data and evaluation of the most appropriate way to use this data, and an analysis of BSTEM that included the methodology incorporated into the program. In addition, comparisons between the results provided by BSTEM and those provided by another stability analysis program, SLOPE/w™, were made in order to determine whether the results from the simpler BSTEM program were comparable to those from other, more complex programs. The potential sources of uncertainty in this analysis were identified, and a sensitivity analysis performed to determine which input parameters had the greatest effect on the amount of erosion and the calculated FS for a given bank. Ultimately, various simulation scenarios were developed and analyzed using the bank stability program, and the results were examined to determine the potential effects of the varied flow releases from Bagnell Dam on bank stability.

1.7. THESIS ORGANIZATION

Section 2 of this thesis includes a review of literature pertaining to erosion, bank stability, methods of analyzing erosion and bank stability, various programs available to calculate erosion and stability, as well as the parameters necessary for a stability analysis and the sensitivity of the analysis to variations in these parameters. Section 3 contains a thorough description of the bank stability model used for this analysis, the methodology and capabilities incorporated into the program, and the limitations of the program. Section 4 describes the modeling approach followed in the execution of this analysis. Section 5 contains a description of the model validation process, and Section 6 provides a discussion on the sensitivity of the results of the analysis to various input parameters. Section 7 includes the results from the various simulation scenarios, and discusses the implications of these results. Section 8 contains conclusions drawn from this analysis, and recommendations for further data collection and potential changes in operating procedures at the dam to minimize impacts on bank stability.

2. REVIEW OF LITERATURE

2.1. GENERAL

Erosion is a naturally-occurring process that is common in most streams and rivers, and can cause changes to river geometry both in its channel patter and in its cross-section. Cross-section changes occur due to the deposition of sediment carried by the flow, which causes an increase in the invert elevation of the river, as well as changes in the geometry of the cross-section. The flow can also cause scour and incision to occur, resulting in a decrease in the invert elevation. Channel pattern changes are typically the result of shear stresses on the riverbanks that are in excess of the shear strength of the bank material, and may result in both shoreline erosion as well as lateral migration of the bank and mass failures [6]. The stability of the bank depends on several variables, including bank geometry, soil type and stratification, pore water pressure, confining pressure provided by the river, and vegetation present on the banks [6].

Streambank materials generally fall into one of two categories: cohesive and noncohesive. Cohesive materials contain large amounts of fine clay particles that tend to have strong chemical and electrochemical inter-particle bonds. Cohesive materials are typically more resistant to erosion than noncohesive materials. This is due in part to the inter-particle bonds, as well as the overall low permeability of the soil. Low permeability results in less seepage, subsurface flow, and piping which can all contribute to bank instability [7]. The accurate determination of the critical shear stress of a cohesive soil is a complex process, as it is dependent on numerous factors that are difficult to quantify, including clay and organic content, and the composition of the pore water [8]. Although less susceptible to erosion, cohesive materials are more likely to fail during rapid drawdown events, as more time is required for the phreatic surface to decrease than is necessary in noncohesive soils [7].

Noncohesive materials include sand, gravel and silt. These soils are more susceptible to erosion than cohesive soils because there are no inter-particle bonds which would help hold the material together, and seepage and piping due to subsurface flow exert forces within the bank towards the river. The flow of the river picks up individual grains as it passes by noncohesive banks, resulting in the banks being eroded grain by grain.

The apparent cohesion of a soil can be increased if the bank is unsaturated, as this causes negative pore pressures, also referred to as matric suction. Matric suction increases the apparent cohesion of the soil as it is an additional force holding the soil together. Apparent cohesion depends not only on the type and structure of soils in a bank, but also on the depth of the phreatic surface within the bank [9].

Although erosion is naturally-occurring, human activities can significantly exacerbate the process, causing more erosion in a shorter period of time. These activities include: increasing the amount of runoff to a river due to an increase in impervious area in the contributing watershed as a result of land development, increasing the amount of sediment in runoff due to land development, increasing the amount of trash and other debris present in runoff, construction of dams, and, the focus of this thesis, releases made from dams to generate hydropower or regulate flood waters.

2.2. CAUSES OF BANK FAILURE

A streambank can be divided into three main sections for the purpose of evaluating failure mechanisms. The toe is located near the invert of the river, and is the section of the bank that is inundated most frequently, making it most susceptible to erosion. The floodplain is the section of the streambank that is only inundated during periods of significant flood flows. Floodplains are typically much flatter than the remainder of the streambank. The main bank is the section between the toe and the floodplain, and is inundated when flows in a river are moderate due to flooding or dam releases. The phreatic surface in the main bank is often at or near the ground surface [7].

There are three main bank failure causes: hydraulic forces, geotechnical properties of the bank which lead to instability, and a combination of hydraulic forces and geotechnical properties. Erosion due to hydraulic forces typically occurs on banks comprised of noncohesive materials [7]. Toe erosion is most common at bends in streams and rivers, where flow is directed toward one of the banks and gravity causes the water to go into a rolling spiral with large velocities in the downward direction. These high velocities can cause the toe to be gradually eroded by a process known as fluvial entrainment, which occurs as the flow directly removes individual soil grains or soil aggregates from banks and transports these particles/aggregates downstream. The amount and rate of erosion of a noncohesive soil depends on the grain size of the soil particles and their distribution within the bank, the shape of the particles, and the density of the particles [10].

Geotechnical instabilities cause bank failure when the downward forces within the streambank are greater than the resisting strength of the soils within the bank. This typically occurs when the streambank is saturated and the flow elevation is rapidly decreasing. Under these conditions the excess moisture in the soil not only adds weight to the bank, but also causes the apparent cohesion of the soil to decrease due to an increase in pore water pressure and a reduction in matric suction. This imbalance of forces ultimately causes a portion of the upper soil mass to be displaced towards the toe of the bank, which is known as mass wasting [7].

The most common failure mechanism is a combination of hydraulic forces and geotechnical instabilities. This type of failure can occur after erosion of the bank toe has caused an increase in the overall bank height or angle, and geotechnical instabilities in the bank become such that mass wasting of the bank occurs [6]. Mass wasting typically occurs during the drawdown phase of a flood while the banks are still saturated, and the confining pressure that was supplied by the flood waters decreases significantly as the flow elevation decreases [10].

Climate-dependent processes occurring within the bank can also affect the amount and rate of erosion. As climate changes occur, the bank can be weakened due to processes such as frost heave and soil desiccation, which weaken the soil, making it more prone to erosion during a large flow event. They can also result in tension cracks, which further weaken the bank, making mass wasting more likely [10].

2.3. FAILURE MECHANISMS

Failure mechanisms typically fall into one of four main categories: planar or slab-type failures, rotational failures, cantilever failures and piping failures [6]. The failure mechanism depends on the geometry as well as the soil type and stratification of the streambank.

Planar or slab-type failures are the most common failure mechanism, and steep streambanks are most susceptible to this type of failure [11]. Steep banks generally fail along a planar slip surface, and the failed block slides toward the toe of the bank before toppling into the channel [6]. It is common for deep tension cracks to appear before a planar failure occurs [12]. An example of a planar or slab-type failure is provided in Figure 2.1.

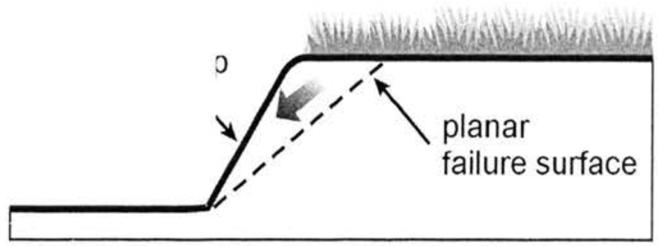

Figure 2.1 Example of Planar Bank Failure [3]

Rotational failures occur along a slightly curved slip surface, and are prevalent in streambanks with large bank heights and mild slopes. Generally a bank with a bank angle of less than 60 degrees is classified as mildly-sloped. The appearance of vertical tension cracks prior to a rotational failure is common. The failure block typically rotates toward the bank while it slides downward [6,12]. An example of a rotational failure is provided in Figure 2.2.

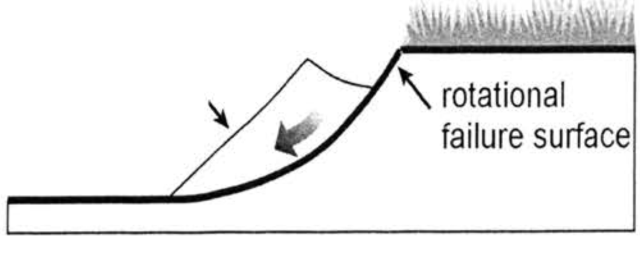

Figure 2.2 Example of Rotational Bank Failure [3]

Cantilever failures are typically seen in streambanks that are composed of a layer of noncohesive material underlying an upper layer of cohesive material. Cantilever, or overhanging, banks form when the bank toe material is noncohesive and is significantly eroded, resulting in a loss of the material that had been supporting the cohesive layer. This loss of material is also referred to as undercutting. When the magnitude of the downward force of gravity on the overhanging bank exceeds its strength, a portion of the bank will break off and fall downward into the eroded area [7]. An example of a cantilever failure is provided in Figure 2.3.

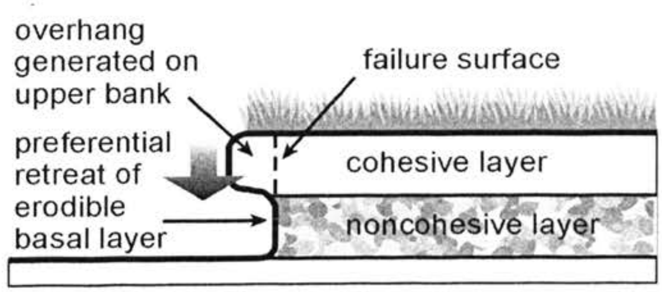

Figure 2.3 Example of Cantilever Bank Failure [3]

A piping failure may occur when the streambank is saturated, and water exfiltrates from the bank into the channel. A piping condition exists when the water carries with it soil particles as it flows toward the river. If the exfiltrating flow is significant enough and removes a sufficient amount of soil, the bank may fail [7].

Ultimately, failure of a streambank occurs when the driving forces downward are greater than resisting forces within the bank. The various forces acting on the failure block include its weight, the weight of the water in the channel acting downward on the block, the hydrostatic force applied by the water in the channel, the hydrostatic force of water in the tension cracks, if they exist, the force of water exfiltrating from the bank, the shear force resisting all downward forces, and the normal force along the failure surface [6].

2.4. PORE PRESSURE

One of the primary factors affecting bank stability is the fluctuation of the groundwater level and corresponding pore water pressure within the streambank. When a bank is saturated, positive pore water pressures cause a reduction in the shear strength of the bank, while simultaneously increasing the weight of the bank, causing a larger downward force to be applied to the failure block. In addition, seepage forces of water exfiltrating from the soil cause further destabilization of the bank. If tension cracks exist and water is present within these cracks, this also decreases the overall stability of the bank due to the forces exerted by the water [8].

While saturated banks tend to cause more bank instability, the absence of water in the upper portion of a bank actually has a stabilizing effect on the bank. When the phreatic surface is located below the ground surface, the upper layer of bank material is unsaturated, and negative pore water pressure exists within the unsaturated material. Negative pore water pressure is also referred to as matric suction due to the increase it causes in the apparent cohesion of the bank material. As the apparent cohesion increases, the shear strength of the bank to resist failure also increases [9,13].

The fluctuation of pore water pressures within a bank is an extremely complex process that is often difficult to predict; however, it has a large impact on the stability of a bank during and after a flow event, which are the periods during which the bank is least stable [14]. When evaluating the stability of a streambank throughout a flood event, it is important to consider the relationship between pore water pressure and the variations of water surface elevation within the channel during a flood hydrograph. On the rising limb of the hydrograph, when flows are increasing within the channel, the phreatic surface within the bank increases as water from the channel infiltrates into the soil. This causes an increase in pore water pressures in the bank, and a subsequent decrease in apparent cohesion and shear strength of the soil as the phreatic surface moves toward the ground surface. Although the pore water pressures increase during the rising limb of the hydrograph, this period is typically the most stable due to the increase in confining pressure that occurs as the water level in the channel rises.

Once the flood peak has passed, the drawdown phase of the hydrograph occurs, resulting in a fairly rapid decrease in confining pressure from the river. The elevation of the phreatic surface does not decrease as quickly as the elevation of the water within the channel; thus a less stable condition develops in which high pore water pressures within the bank are coupled with low confining pressures from the river. If the instability of the bank is sufficient, failure may occur during this phase of the flood hydrograph.

It has been widely accepted that the highest probability of bank failure occurs after the peak of the flood hydrograph has passed, and the water level in the channel is decreasing [13]. The difficulty in determining the actual likelihood and timing of bank failure lies in the complexity of the water table drawdown process, and the relative inability to quantitatively define the relationship between the elevation of the phreatic surface (pore water pressures) and flow elevation (confining pressure of river) in the channel, due largely to difficulties related to monitoring these parameters [14]. The issue is further complicated by the fact that the depth to the phreatic surface is not influenced only by the flow elevation within the channel, but also varies with seasonal fluctuations of rainfall and evapotranspiration [13].

The relationship between the elevation of the phreatic surface within the bank and water surface elevation in the channel is particularly important in regulated rivers, where high flows are generally sustained for long periods of time, followed by a rapid decrease in flow [9]. The extended duration of high flows generally results in near or complete saturation of the banks, maximizing pore water pressures. When the flow is abruptly decreased, the water surface elevation and subsequently the confining pressure of the river decrease quickly. The phreatic surface elevation decreases at a significantly slower rate than the water surface elevation in the stream due to the relatively slow movement of water through the soil, establishing favorable conditions for bank failure [9].

The uncertainties involved in determining pore water pressures within a streambank were also discussed in the 2003 erosion analysis, which stated that it is "very difficult or nearly impossible to define a typical groundwater flow pattern for the different stages of the river" [5]. Although it is possible to make generalizations regarding the rate of drawdown of the water table in various soil types, it is difficult to quantify these generalizations. The report also pointed out that most of the uncertainties related to modeling bank stability during a flood event occur during the drawdown phase of the flood, when pore pressures have the largest impact on stability but are difficult to accurately enumerate [5]. It is because of these uncertainties that many streambank stability analyses assume that the phreatic surface remains at the ground elevation throughout the entire flood hydrograph, as this is the most conservative assumption.

2.5. VEGETATION EFFECT ON STABILITY

The presence or lack of vegetation on streambanks, as well as the type of vegetation, have an effect on stability. The impact of vegetation is difficult to quantify as it is a complex issue that varies from streambank to streambank. In most cases, vegetation is considered to have a positive effect on streambank stability; however, it is possible for it to have a negative impact under certain conditions [6].

Vegetation intercepts some precipitation, reducing the amount of water that infiltrates into the ground, as well as reducing displacement of soil particles due to the impact of the precipitation [8]. In addition, vegetation roots have significant tensile strength and provide reinforcement to the soil, absorbing a portion of the total shear stress that is applied to the soil. In addition, the phreatic surface in a vegetated streambank is generally lower than in non-vegetated banks as the vegetation intercepts some rainfall, and the roots also remove moisture from the soils [6]. It is estimated that the critical shear stress of a streambank may increase by up to a factor of three when sufficient vegetation is present [11].

Vegetation also decreases temperature fluctuations in the soil, which can help prevent or reduce frost heave and desiccation that are known to contribute to bank instability. It also causes the roughness of the banks to increase, which directs flow more toward the center of the channel, decreasing shear stresses on the banks [8].

Vegetation may also, however, contribute to instability of a bank if the roots create seepage paths. Seepage paths increase the amount of water in the soil, increasing pore water pressure, and potentially resulting in piping [6].

2.6. OVERVIEW OF STABILITY ANALYSIS

There are various methods of evaluating bank stability in a river, the most common of which is the limit equilibrium method. This method calculates a FS for a streambank under the specified conditions. The FS is a ratio of the resisting forces of a streambank to the driving downward forces applied to the bank [6]. This is equivalent to the factor by which the shear strength of the bank must be reduced in order for the resisting and driving forces to be in equilibrium [6].

Many studies that attempt to determine bank stability simplify the calculations by assuming only planar failures are possible, and by avoiding the determination of a relationship between pore water pressures and confining pressures. In these studies two soil conditions are typically evaluated: dry conditions, where the phreatic surface is assumed to be at or slightly above the flow elevation in the channel, and worst case conditions, which assumes total saturation of the bank followed by a rapid drawdown of the water level in the channel [13]. These assumptions and simplifications are made because the use of traditional limit equilibrium methods to determine bank stability is limited due to the difficulties in determining pore water pressures within the bank [12].

Other studies have attempted to estimate the fluctuations of the pore water pressures in a bank during a flood event. Rinaldi et al. (2004) determined that it was necessary to determine the effects of the flow release scenarios on pore water pressures and erosion of the bank toe in order to accurately calculate the stability of the bank [13]. This was accomplished by coupling two models. First, a two-dimensional finite element hydrologic model called GeoSlope SEEP/W™ was used to simulate water movement through the bank and the associated pore pressures. This data was then input into a bank stability model to determine the FS. When a detailed seepage analysis is performed as in this study, it is necessary to accurately define initial soil moisture conditions. This study used average spring water table elevations for the area of interest, and included a small amount of evaporation in order to establish reasonable initial moisture conditions [9]. Although this approach likely provides for a more realistic stability analysis than in those analyses that make general assumptions regarding pore water pressures, it also requires significantly more data before the analysis can begin. In addition, with the increase in data necessary to run a model comes an increase in the uncertainties associated with the data, which could lead to a decrease in the confidence of the accuracy of the results.

In the absence of sufficient groundwater table elevation data, the most common approach to a stability analysis is to assume a rapid drawdown scenario, which is common in rivers with fluctuating flows [15]. This is also the most conservative method of analyzing bank stability, as pore water pressures are assumed to be maximized.

3. BANK STABILITY AND TOE EROSION MODEL (BSTEM) DESCRIPTION

3.1. GENERAL

The calculations of the amount of erosion and FS for all outflow scenarios evaluated during the course of this study were carried out with version 5.2 of BSTEM. The user must enter several pieces of information describing the streambank of interest before the model can be run. Station-elevation data describing the geometry of the cross-section of interest must be provided, as well as the elevation of the top of the bank toe. The user must also enter the thicknesses of and the materials comprising the various soil layers. The material of each layer may be entered in one of two ways: the type of material may be selected from a drop-down menu of materials preset within the program, or the parameters describing the material may be entered numerically. If the material is selected from the drop-down menu, the program assumes typical values for each of the parameters. The parameters used to describe the soil include the friction angle (φ' in degrees), the cohesion (c' in kPa), the saturated unit weight (γ in kN/m^3), the critical shear stress (τ_c in Pa), the erodibility coefficient (k in cm^3/Ns), and an angle that describes the increase in shear strength due to matric suction (φ^b in degrees).

Once the cross-section properties have been entered into the model, it is necessary to indicate the type of vegetation and bank and toe protection, if there is any present. The depth to the phreatic surface must also be provided to obtain the pore pressures within the bank, or the user has the option to manually specify pore pressures for each layer. Other input requirements include the length and slope of the reach in which the cross-section is located, as well as the elevation of flow within the cross-section and the duration the specified flow elevation is maintained.

The program then uses all of the provided data to calculate the amount of toe erosion that occurs and the minimum FS for the given conditions.

3.2. METHODOLOGY

BSTEM is capable of evaluating the erosive effects of flow through a cross-section by calculating the amount of material eroded away due to hydraulic forces (shear stresses) of the flow. In addition, it can determine the stability of the bank via the calculation of a FS for the specified scenario assuming a planar failure of the soil bank. It is also possible to use the modified cross-section geometry determined through the toe erosion calculations in the bank stability analysis. By using this option the cumulative effects of erosion and undercutting of the cross-section on the FS throughout a flood hydrograph can be computed [16].

The amount of eroded material is calculated by comparing the critical shear stress of the soil with the shear stress applied by the streamflow. This is determined through the use of the following equation:

$$\tau_0 = \gamma_w RS \tag{1}$$

where τ_0 is the calculated average boundary shear stress exerted by the flow in Pascals (Pa), γ_w is the unit weight of water equal to 9.801 kN/m^3, R is the hydraulic radius of the channel cross-section for the given water surface elevation in meters (m), and S is the average channel gradient in m/m. The hydraulic radius of a channel is typically considered to be equal to the water depth in channels that are considered wide. Figure 3.1 illustrates how the shear stress is distributed in BSTEM [16].

The shear strength of the soil is determined using one of two criteria, depending on whether the bank is saturated or unsaturated. In saturated soils, the Mohr-Coulomb criterion is used to calculate the shear strength according to the following equation:

$$\tau_f = c' + (\sigma - \mu_w)\tan(\varphi') \tag{2}$$

where τ_f is the shear strength of the soil, c' is the effective cohesion of the soil, σ is the normal stress applied to the boundary, μ_w is the pore water pressure, and φ' is the effective angle of internal friction. φ' is measured in degrees, and all other variables are measured in kPa [3].

In the case of unsaturated soils where negative pore pressures exist, the Fredlund et al. (1978) criterion is used to determine the shear strength of the soil by the equation [8]:

$$\tau = c' + (\sigma - \sigma_a)\tan(\varphi') + (u_a - u_w)\tan(\varphi^b) \tag{3}$$

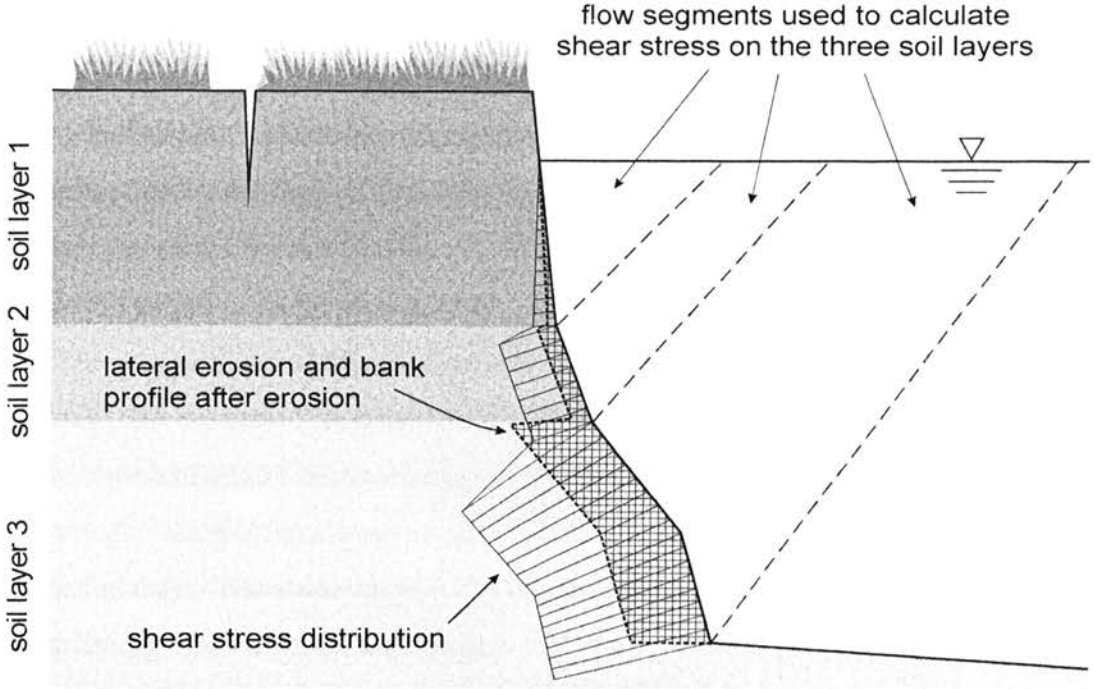

Figure 3.1 Example of Shear Stress Distribution in BSTEM [3]

In this equation, τ is the shear strength of the soil, c' is the effective cohesion of the soil, σ is the normal stress on the failure block, u_a is the pore air pressure, φ' is the friction angle in terms of effective stress, u_w is the pore water pressure, and φ^b is the angle that represents the increase in apparent cohesion due to negative pore pressures. The angles are measured in degrees in this equation, and all other variables are measured in kPa [8,13].

The longer the given flow elevation is sustained within a cross-section, the greater the amount of material that will be eroded. An area of eroded material is determined for each cross-section; this can be converted to a volume of eroded material by multiplying by the reach length. Once the eroded profile for the given flow elevation and duration has been determined, the FS for a planar failure of the bank is computed using the limit equilibrium method.

The pore pressures within a bank are a crucial factor in the determination of the FS. As discussed previously, positive and negative pore pressures play a large role in the overall stability of a streambank. In BSTEM, if pore pressures are not defined for each soil layer and the user simply provides a depth to the phreatic surface, pore pressures are calculated for each layer. The pore pressures below the phreatic surface are calculated assuming hydrostatic conditions, thus the pore pressure is equal to the unit weight of water multiplied by the head of the water table above the centerline of each soil layer. Pore pressures above the water table are calculated the same way; however the pore pressures are negative rather than positive, and represent matric suction.

The program has a built-in algorithm that iterates over multiple failure scenarios to determine the scenario that results in the lowest FS. The iterations cover many different combinations of shear emergence elevations and shear angles. The shear emergence elevation is the elevation on the bank where the failure plane will intersect the cross-section face. The shear angle is the angle of the failure plane. Figure 3.2 illustrates the shear emergence elevation and angle.

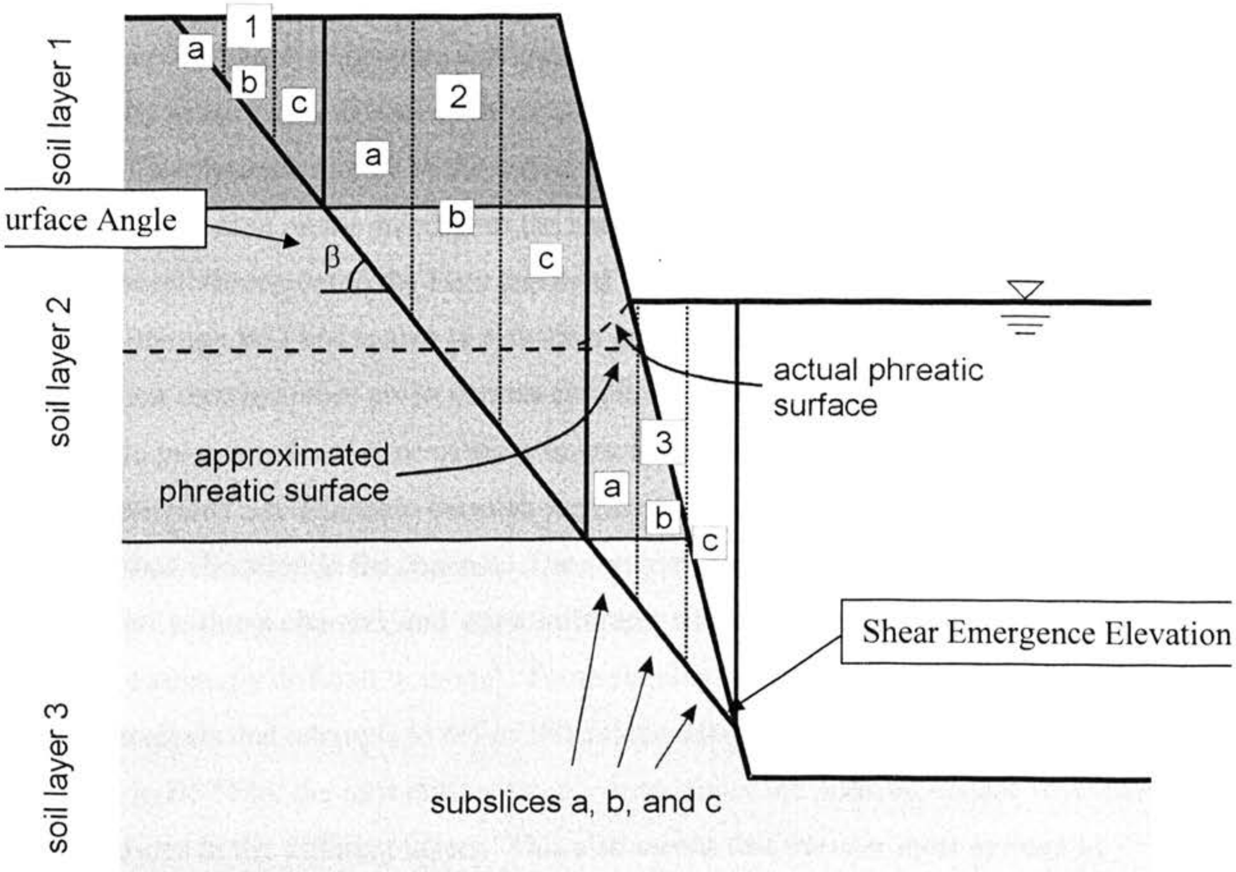

Figure 3.2 Figure Showing Shear Emergence Elevation and Shear Surface Angle [3]

If an analysis results in a FS less than 1.0, the bank is considered *unstable*. If the FS is between 1.0 and 1.3, the bank is considered *conditionally stable*. This is because, although theoretically a FS of greater than 1.0 should indicate that the shear strength of the soil is exactly equal to the shear stresses exerted on the failure block, and thus the bank should not fail, there is inherent uncertainty and error in the calculation of the FS. A FS of greater than 1.3 indicates that the bank is considered *stable*, and should not fail under the given conditions.

3.3. MODEL CAPABILITIES AND LIMITATIONS

BSTEM is capable of evaluating varying and complex bank geometries composed of up to five different soil layers, as well as the bank toe. With respect to bank toe erosion, BSTEM is capable of calculating the shear stresses exerted by the flow on the bank toe and the resulting erosion [3]. Each of the five soil layers may have completely separate properties from the other layers, which results in the ability of the user to evaluate composite banks with soils of varying strengths. The model is also able to incorporate the effects of positive and negative pore water pressures within the bank on the stability of the bank, as well as the confining pressure provided by the flow in the channel. One optional feature of the model is the ability to model the reinforcement effects of vegetation on the stability of the bank. In addition, the user is also able to add man-made reinforcement to the bank and bank toe if appropriate [3].

Although BSTEM is able to provide a general approximation of the stability of various bank stratigraphies given various conditions, there are some model limitations that should be pointed out. One of the primary limitations of the model is that there is no way to determine a relationship between the elevation of the phreatic surface and the water surface elevation in the channel. The complex processes that occur as water levels rise and fall within a channel, and water infiltrates and exfiltrates into and out of the bank, are extremely difficult to model. Some stability analysis programs include a seepage analysis that attempts to define this relationship. In order to account for pore pressures in BSTEM, the user must manually enter either the phreatic surface elevation or pore pressures in the different layers. This also means that the user must attempt to determine the location of the phreatic surface and its fluctuations during a flood event. This results in a significant amount of uncertainty in the stability analysis.

BSTEM also assumes that the water table is horizontal within a particular bank, when in fact it varies across the bank depending on the proximity to the river. Figure 3.2 above shows how the assumed horizontal water table differs from the actual water table.

One additional limitation of the model is that it only assumes planar or cantilever failures of a bank when carrying out the stability analysis. This means that rotational failures and piping failures cannot be considered while running this model. The assumption that failures are either planar or cantilever failures is not unrealistic, as planar failures are the most common form of streambank failure encountered.

A final limitation of the model is that it is unable to automatically provide an analysis of bank stability throughout a hydrograph. This limitation can be overcome by the user performing numerous calculations throughout each flood hydrograph. The calculations must be cyclical in that the user must perform an erosion analysis for one water surface elevation, export the eroded profile for use in the stability analysis for that same water surface elevation, and then repeat the process for all remaining water surface elevations of the hydrograph.

4. MODELING APPROACH

The modeling approach consisted of several steps, the first of which involved setting up a separate BSTEM model for each of the cross-sections. For each cross-section the bank geometry was specified, as well as the thickness and properties of the various soil layers. At that point it was possible to begin calculating the erosion amounts and FSs for each cross-section, and their fluctuations throughout the various outflow scenarios.

For reasons previously discussed, it was necessary to make some assumptions regarding the elevation and fluctuation of the phreatic surface in the streambank throughout each flow scenario. The "worst case" scenario was evaluated for all cross-sections, and assumed that the phreatic surface did not fluctuate, but remained at the top of the ground surface throughout the entire flood event. This scenario is most critical in terms of bank stability because this is the case in which pore water pressures in the bank are maximized.

The "best case" scenario assumed that the water table fluctuated slightly throughout the flood event. For this scenario, it was assumed that if the flow elevation in the channel increased, the phreatic surface elevation increased accordingly so that it was equal to the water surface elevation for each time step; however, it decreased at a rate equal to approximately one-tenth the rate that the flow elevation in the channel decreased. This scenario was considered the "best case" scenario as the bank is not assumed to be completely saturated at any point throughout the hydrographs, thus pore pressures will never be maximized. Due to capillary action and infiltration from precipitation, the water table in a bank typically is higher than the elevation of the water surface in the channel. The purpose of evaluating both of these drawdown scenarios was to determine how erosion and the FS would be affected by the theoretical maximum range of water table elevations that could realistically occur within a given bank.

The results of the 2001 hydrodynamic model provided water surface elevations at each cross-section for each outflow hydrograph at three-hour time intervals. Due to the nature of BSTEM, which requires manual entry of each individual water surface elevation and phreatic surface elevation, and the exceptionally large number of data points for each cross-section, the erosion and FS were calculated every 12 or 24 hours rather than every three hours. This allowed for the overall effects of the flow fluctuations to be accounted for, while reducing the amount of data points to a more manageable number.

The total amount of erosion and average erosion rate for all six outflow scenarios at each cross-section were determined. Upon completion of these calculations, the three outflow hydrographs that caused the greatest amount of erosion and average erosion rates were identified. The FSs throughout these three hydrographs were then evaluated at all of the cross-sections. The reason for this simplification was also due to the time-consuming nature of data entry in the BSTEM model. Rather than spend an excessive amount of time evaluating all six hydrographs at all the cross-sections, the three most critical hydrographs were evaluated at each cross-section. This allowed for a more thorough investigation of the effects of flow fluctuations on erosion and bank stability. One cross-section, cross-section 2, was selected for a more in-depth analysis. At this cross-section, all six outflow hydrographs were evaluated, stability analyses omitting toe erosion were completed, and various groundwater drawdown assumptions were considered.

It was assumed in the calculations for each water surface elevation that the flow elevation was sustained within the cross-section for 12 or 24 hours, depending on which time step was selected for the given scenario. This assumption coupled with all of the other data already entered into the program allowed for the calculation of the average applied boundary shear stress, and determined the eroded area of the bank and toe for each time step. The eroded cross-section was then exported into the input geometry and this new cross-section was used to determine the FS for the given water surface elevation. This iterative process was completed for every water surface elevation throughout each of the outflow hydrograph scenarios evaluated.

Each outflow scenario was assumed to be an independent event; thus the original cross-section as reported in the 2003 erosion analysis was assumed to be intact at the beginning of each outflow scenario. The purpose of this was to determine the individual effects of each flow scenario on the amount and rate of erosion and the stability of the cross-sections. Also, since the flow patterns are cyclical, there was no way to determine which flow scenario should be evaluated first if the eroded cross-sections were to be carried over for the subsequent flow pattern.

The erosion area and FS resulting from each water surface elevation during each outflow scenario were plotted on a graph with the appropriate outflow pattern to determine the relationship, if any, that existed between outflow and the FS. The total amount of eroded material during each outflow scenario for each cross-section was also determined and plotted to demonstrate the outflow pattern that resulted in the most erosion.

Two additional analyses were also completed to provide additional data regarding the effect on the banks of the outflow scenarios. The first was completed to determine whether the slope of the outflow hydrograph had a significant impact on the stability of the bank. In other words, would a decrease in the time step of the outflow hydrograph result in a measurable increase or decrease in the erosion amount and FS? This determination was made by running additional analyses for cross-section 2 for one outflow hydrograph in which the time step was decreased from 24 hours to 12, 6 and 3 hours.

The second additional analysis was completed to determine whether the omission of toe erosion would significantly affect the stability analysis. In this portion of the study, a strictly stability analysis was performed for all of the cross-sections; no toe erosion was assumed to occur during the outflow hydrographs.

The results of these analyses could potentially be used in the future to recommend outflow scenarios from Bagnell Dam that would result in less bank erosion and instability than current typical outflow scenarios.

5. MODEL VALIDATION

5.1. GENERAL

The model used for this study, BSTEM, makes many simplifying assumptions in the calculation of the FS. In an effort to determine whether these assumptions result in reliable FSs, input data from a stability analysis completed for a streambank of the Sieve River in Tuscany (Italy) was entered into BSTEM, and the results calculated therein were compared to those reported in Rinaldi et al. (2004), "Monitoring and Modelling of Pore Water Pressure Changes and Riverbank Stability During Flow Events" [13].

The results of Rinaldi et al. (2004) were selected for use in the validation process for a number of reasons. The primary reason for this selection was that the stability analysis was determined through a limit equilibrium method, and bank failures were limited to planar failures. These are consistent with the analysis methods of BSTEM, which would remove the possibility of discrepancies in results due to methodology differences. Also, the study incorporated a seepage analysis, allowing for changes in pore pressures within the bank to be approximated and more accurately represented in the model. In addition, the model results were compared with actual field data to determine their accuracy, verifying that the model used produces reasonably accurate results.

5.2. STUDY OVERVIEW

The focus of the study was the Sieve River, which is a naturally-eroding tributary of the Arno River located in Tuscany, central Italy. Monitoring took place from February 1996 through February 2000 to collect data regarding erosion and bank instability along the Sieve River, and how these related to changes in river flows. The monitoring also included the use of piezometers and tensiometers to determine the groundwater table elevation and matric suction profile in the bank. The main purpose of this study was to determine how pore water pressures vary within the streambank, and their corresponding effects on the overall stability of the bank. A secondary objective included demonstrating the use of a coupled modeling system which incorporates both a seepage analysis to determine pore pressures, and a limit equilibrium analysis for bank stability. One final objective was to evaluate the response of the FS of the bank to various flow events.

The pore water pressures were calculated using a program called SEEP/w™, which uses Darcy's Law to determine the velocity of groundwater flow within a saturated streambank, and estimates hydraulic conductivity of unsaturated banks from the water content and pore water pressure in the soil. The estimations of groundwater flow velocities coupled with the principle of conservation of mass are then used to perform a two-dimensional, finite element seepage analysis.

The results of the seepage analysis were used as input in the stability analysis, which was completed using the software SLOPE/w™. This was an iterative process, requiring that the seepage analysis be completed for each time step, and this data entered into the stability analysis at each time step. Data from the piezometers and tensiometers installed along the bank was used to define the initial groundwater conditions and matric suction profile in SEEP/w™ prior to the beginning of a flow event. In cases where the flow event occurred prior to the installation of the piezometers and tensiometers, the initial water table elevation was assumed to be equal to the water surface elevation within the channel prior to the beginning of the flow event.

The primary limitation of the stability analysis in Rinaldi et al. (2004) is that it did not account for bank toe erosion and undercutting which contribute greatly to bank instability. The geometry of the bank was assumed to remain intact throughout the entire simulation, and only mass failures of the upper bank were considered as failure mechanisms. Based on the field observations, mass failure was the primary failure mechanism along the Sieve River; thus this limitation was not considered to affect the accuracy of the results.

5.3. RIVERBANK PROPERTIES

Detailed data regarding the properties of the riverbank used in the analysis was provided. This data was used unchanged as input into BSTEM for the model validation. Figure 5.1 is an illustration of the stratigraphy of the bank as provided in Rinaldi et al. (2004).

The riverbank was divided into five distinct soil layers, *a* through *e*, each with a corresponding description and geotechnical properties. Layer *a* is 1.2 m thick and is described as massive silty fine sand. Layer *b* is 0.75 m thick, and consists of sand in the upper portion of the layer, and cobbles in the remaining lower portion. Layer *c* is 1.15 m

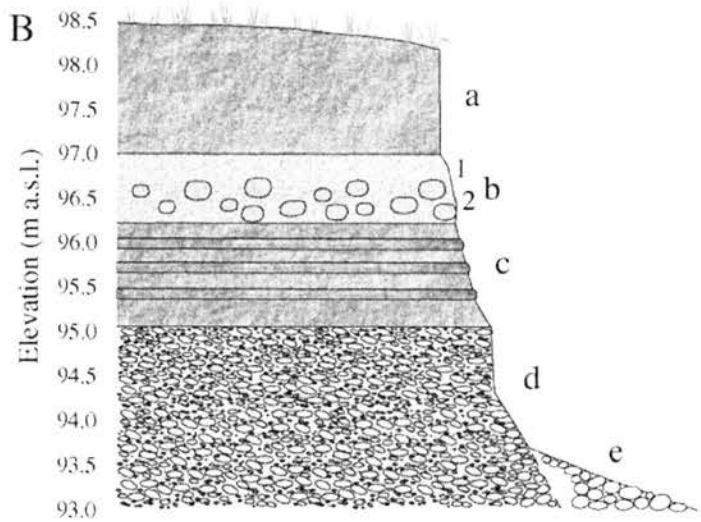

Figure 5.1 Bank Stratigraphy [13]

thick, and is comprised of silty sand, with silt sublayers throughout the upper and central portions of the layer. Layer *d* is a 2.0 m thick layer of packed and imbricated sand, gravel and cobbles. Layer *e* is the bank toe, and the soil type in this layer is described as loosely packed gravel and cobbles.

The geotechnical properties for layers *a* through *c* were determined through a series of triaxial tests and borehole shear tests. The values reported for these layers are provided in Table 5.1, and were used unchanged in the BSTEM model. In cases where the triaxial tests and borehole shear tests resulted in slightly different parameter values, the values were averaged before being used as input.

Table 5.1 Geotechnical Properties

Layer	φ' (degrees)	c' (kPa)	γ (kN/m3)	φ_b (degrees)
a	34	2	17.7	28
b	37.5	1	18.3	32
c	34.5	2	17.8	26

Exact geotechnical properties were not provided for layers *d* and *e*. For these layers, the default parameters in BSTEM for the specified soil type were used. Based on the soil descriptions, a range of parameter values was possible for layers *d* and *e*. Due to a lack of more specific data, the more conservative parameter values were adopted.

5.4. ADDITIONAL INPUT DATA

A plot was provided in Rinaldi et al. (2004) that depicted the variations in the FS as compared to the water table elevation and the elevation of the water surface in the channel. Twenty-four time steps were used to evaluate a flood event that lasted from noon on December 13, 1996 through 6:00 pm on December 17. Numerical values of the FS for each time step were provided in the paper, but only a graphical representation of the water table elevations and river water surface elevations were available. Estimates of these parameters at each time step were made from the plotted data. The mean channel gradient was also given in the paper as 0.003; this value was used in the BSTEM model.

5.5. RESULTS COMPARISON

The FS was calculated in BSTEM at each of the 24 time steps, and the results were plotted on a graph alongside the FSs reported in Rinaldi et al. (2004). Figure 5.2 below shows the fluctuations of the FSs and how they compare with one another.

The FSs calculated in BSTEM and their variations throughout the flood event were comparable to those reported in Rinaldi et al. (2004), indicating that BSTEM provides very similar results to Slope\w™. The maximum percentage difference between the Rinaldi FSs and those calculated in BSTEM was 17.2 %; however, the average percentage difference was only 6.3 %. The differences in the results could potentially be attributed to the error incurred during the estimation of water table elevation and water surface elevation from the figure in the report. Another potential source of the discrepancies in the results could be related to a modeling component that differs between the models. For the model used in Rinaldi et al. (2004), the actual pore pressures in each soil layer obtained through the seepage analysis were input into the stability analysis. In BSTEM, the phreatic surface elevation was entered and the pore pressures were calculated by the program. BSTEM assumes the pore pressures vary linearly with bank height, whereas the pore pressures obtained in the seepage analysis

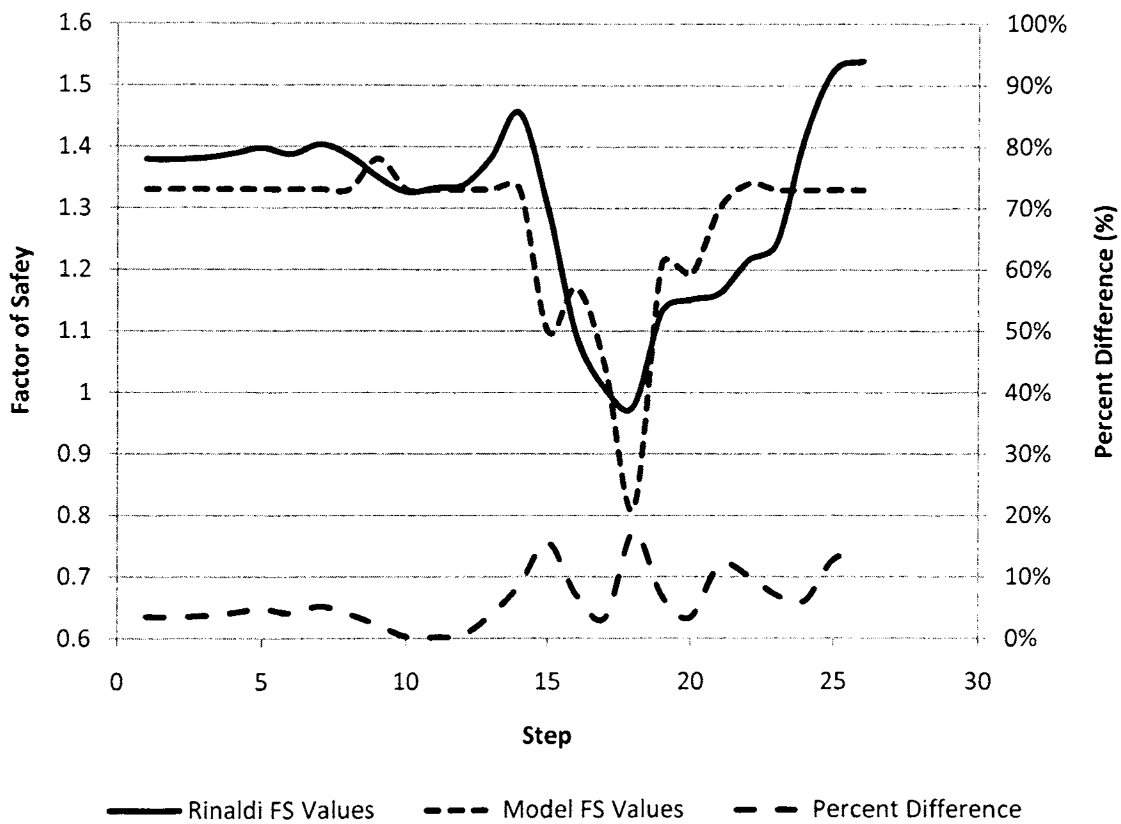

Figure 5.2 Validation Comparison Plot

likely were not distributed linearly. This slight difference could cause the discrepancies seen in the comparative plot above.

The general shapes of the plots are also very similar, meaning that the effects of the fluctuations in water table elevation and water surface elevation are represented fairly consistently in both stability models. In addition, BSTEM does not appear to consistently over or underestimate the FS.

Overall, the results calculated during the validation process indicated that BSTEM is capable of providing estimates of FSs for a composite bank throughout a flood event comparable to those calculated in other stability models. The accuracy of the results, however, is still dependent on the quality of field data used as input to the model.

5.6. STUDY CONCLUSIONS

Based on the results of the simulations, Rinaldi et al. (2004) made several observations regarding bank stability during flood events. One such observation was that hydrographs with only one distinct peak, rather than numerous fluctuations in flow throughout the flood event, resulted in greater bank stability. An additional observation was that it is not necessary for a bank to be completely saturated for a failure to be triggered. As pore pressures began to increase, and the apparent cohesion of the bank to decrease, the FS was sometimes low enough to indicate that a mass failure was possible, even if the bank was not fully saturated. This scenario was most common in streambanks composed of mostly silts and sands.

6. SENSITIVITY ANALYSIS

6.1. GENERAL

It is very difficult to accurately characterize the geotechnical properties of a composite bank and how they vary over the bank height; however, defining parameters for each of the bank materials is an integral step in using BSTEM to perform a stability analysis. The appropriate values for each geotechnical parameter vary according to soil type, soil compaction, amount of organic material mixed into the soil, as well as other factors [10]. It is important to accurately characterize the various soil strata within a bank because the results of the BSTEM erosion calculations and stability analysis are only as accurate as the available input data.

A sensitivity analysis was performed to determine the quantitative effect of varying parameter values on the resulting erosion amounts and FS. The purpose of this was to determine which parameters had the greatest impact on the results of the analyses, and to thus provide a basis for further geotechnical investigations if it is desired that the BSTEM model of the Osage River to be further refined.

6.2. AVAILABLE DATA

The geotechnical data collected for the 2003 erosion analysis included detailed boring logs at each of the cross-sections, which provided an accurate description of the soil types at various depths within the bank; however, no geotechnical parameters necessary for use in BSTEM were directly measured. Because of this it was necessary to assume reasonable values for the parameters for each cross-section. There is an option in BSTEM where the user selects a general description of each soil layer, and typical values for each of the parameters are used in the erosion calculations and stability analyses. Due to the lack of more specific data, this option was chosen to describe the bank materials.

6.3. APPROACH

Cross-section 2 was selected for use in the sensitivity analysis because it has a relatively mild bank angle, and provided reasonable results for all of the outflow scenarios. In addition, several types of soil were represented in this cross-section, including stiff clay, soft clay, silt and angular sand. The bank geometry and stratigraphy of cross-section 2 are depicted in Figure 6.1 below.

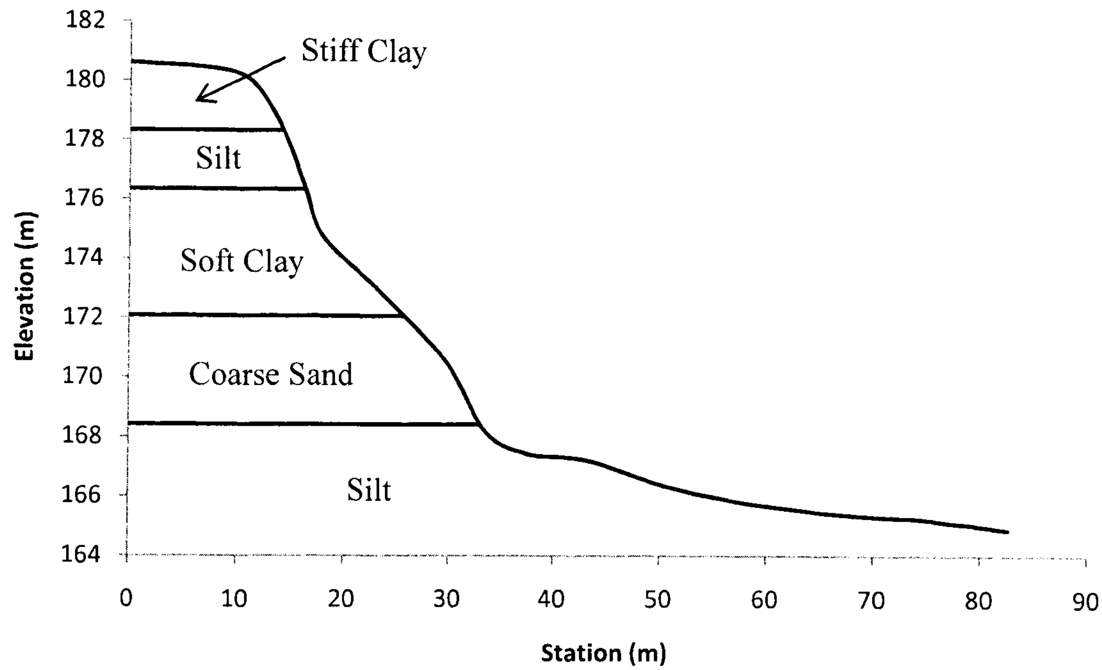

Figure 6.1 Cross-Section 2 Geometry and Stratigraphy

The High Flow scenario was chosen as the outflow scenario to be used in the sensitivity analysis. This outflow hydrograph was selected because it resulted in a large range of water surface elevations at cross-section 2, and also contained periods of rapid drawdown and rising water surface elevations.

Twenty-four individual sensitivity analyses were completed, each using default values for all of the parameters except for one. The parameters were varied individually by both increasing and decreasing the default value by 10 and 20 %. This is similar to the sensitivity analysis completed in Jha et al. (2005), which changed the base parameter values by 5 and 10 percent to determine the effect on the stability analysis outcome [11]. The parameters that were adjusted for this sensitivity analysis included φ', c', γ, τ_c, k, and φ^b, which were defined in Section 3.1. Table 6.1 contains a list of the various sensitivity analyses performed and a description of the parameter and percentage increase or decrease used in each analysis.

The amount of erosion and stability of cross-section 2 were determined for each 24-hour time step of the High Flow scenario. This entire process was completed for each

Table 6.1 Description of Various Sensitivity Analyses

Sensitivity Analysis	Description	Sensitivity Analysis	Description
1	φ' decreased by 10%	13	φ' decreased by 20%
2	φ' increased by 10%	14	φ' increased by 20%
3	c' decreased by 10%	15	c' decreased by 20%
4	c' increased by 10%	16	c' increased by 20%
5	γ decreased by 10%	17	γ decreased by 20%
6	γ increased by 10%	18	γ increased by 20%
7	φ_b decreased by 10%	19	φ_b decreased by 10%
8	φ_b increased by 10%	20	φ_b increased by 10%
9	τ_c decreased by 10%	21	τ_c decreased by 20%
10	τ_c increased by 10%	22	τ_c increased by 20%
11	k decreased by 10%	23	k decreased by 20%
12	k increased by 10%	24	k increased by 20%

of the 24 sensitivity analyses. Only one parameter value was adjusted at a time so that the relative impact of each parameter on the outcome of the analysis could be determined. The sensitivity of the model to changes in bank geometry were not included in the sensitivity analysis, as the survey data collected for the 2003 erosion analysis was very detailed and considered to be an accurate depiction of the streambank at each cross-section location.

The phreatic surface was assumed to be located at the ground surface for all analyses. This is the most critical water table condition for the stability of the streambank, and it was held at as a constant so that only the changes to parameter values would contribute to changes in the outcome of the stability analysis. Table 6.2 contains a list of the soil types found in cross-section 2, and the default parameter values for each type. These values were increased and decreased by 10 and 20% for the sensitivity analysis.

In addition to varying the parameter values, the timing of the hydrograph was also decreased by various factors to determine whether this would have a significant effect on

Table 6.2 Default Parameter Values for Soil Layers in Cross-Section 2

Soil Type	φ' (deg)	c' (kPa)	γ (kN/m^3)	φ_b (deg)	τc (Pa)	k (cm^3/Ns)
Stiff clay, moderate cohesive	20	15	18	15	5.0	0.045
Silt, moderate cohesive	30	3	18	15	5.0	0.045
Soft clay, erodible cohesive	25	10	18	15	0.1	0.316
Angular sand, coarse sand	36	0	18	15	0.5	0.141

the outcome of the erosion calculations and stability analyses. The selected outflow hydrograph was the High Flow scenario, and the variation in water surface elevation at cross-section 2 during this hydrograph is shown in Figure 6.2 below. Also shown in this Figure are the shortened hydrographs used in this phase of the analysis.

The purpose of shortening the original stage hydrograph was to determine whether the rate of increase and/or decrease of the hydrograph would have a significant effect on the total erosion of the bank or the bank stability. This is especially important in the drawdown portion of the hydrograph, when the confining pressure of the river decreases suddenly, and the bank remains saturated.

6.4. RESULTS

The area of eroded material for each sensitivity analysis was compared with those obtained using all default values for the soils, and a percent increase or decrease caused by varying each parameter was calculated. Figures 6.3 and 6.4 are bar graphs which allow for a visual comparison of the impacts of each parameter on the amount of erosion. As can be seen from these figures, the two parameters which appear to have the most significant effect on the amount of area eroded from the cross-section are the critical shear stress and the erodibility coefficient of the soil, with variations of the erodibility resulting in the greatest differences. The other parameters do not significantly affect the amount of erosion, increasing it by zero to 0.10%. This is a reasonable finding, as the critical shear stress and erodibility coefficient are the main factors used in the calculation of the amount of erosion.

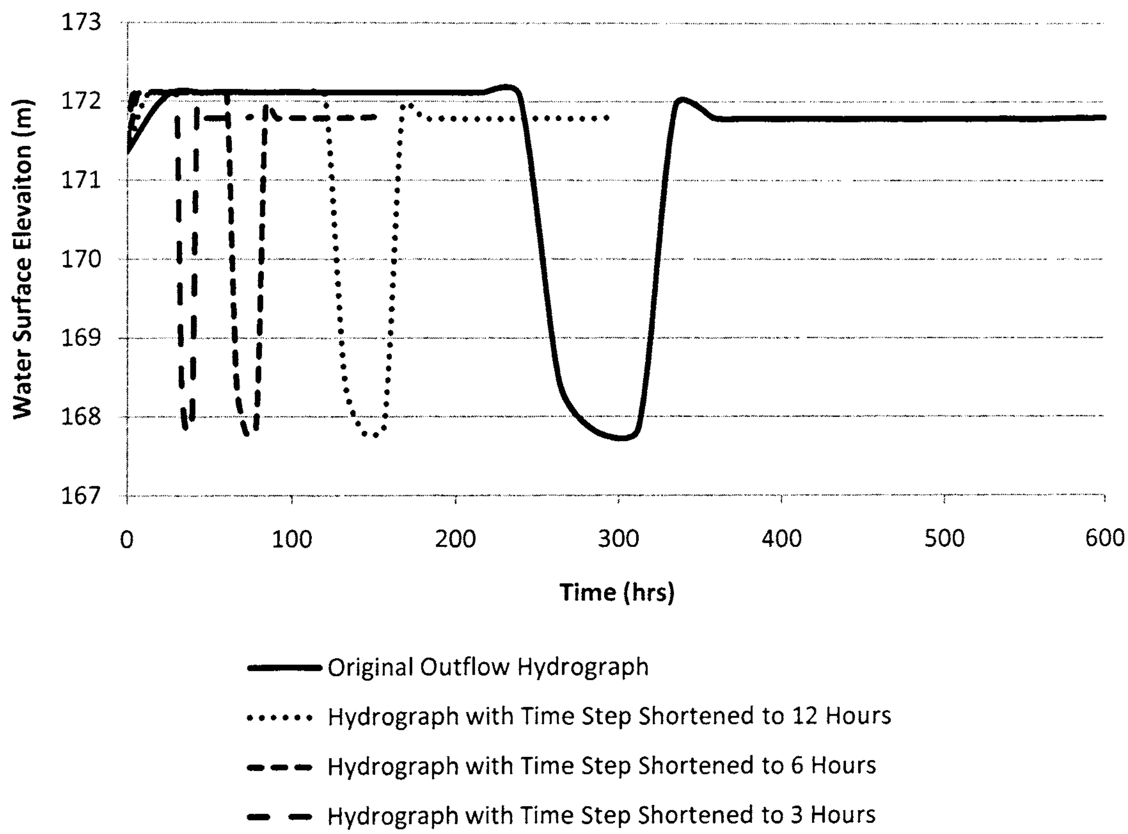

Figure 6.2 High Flow Scenario and Shortened Hydrographs

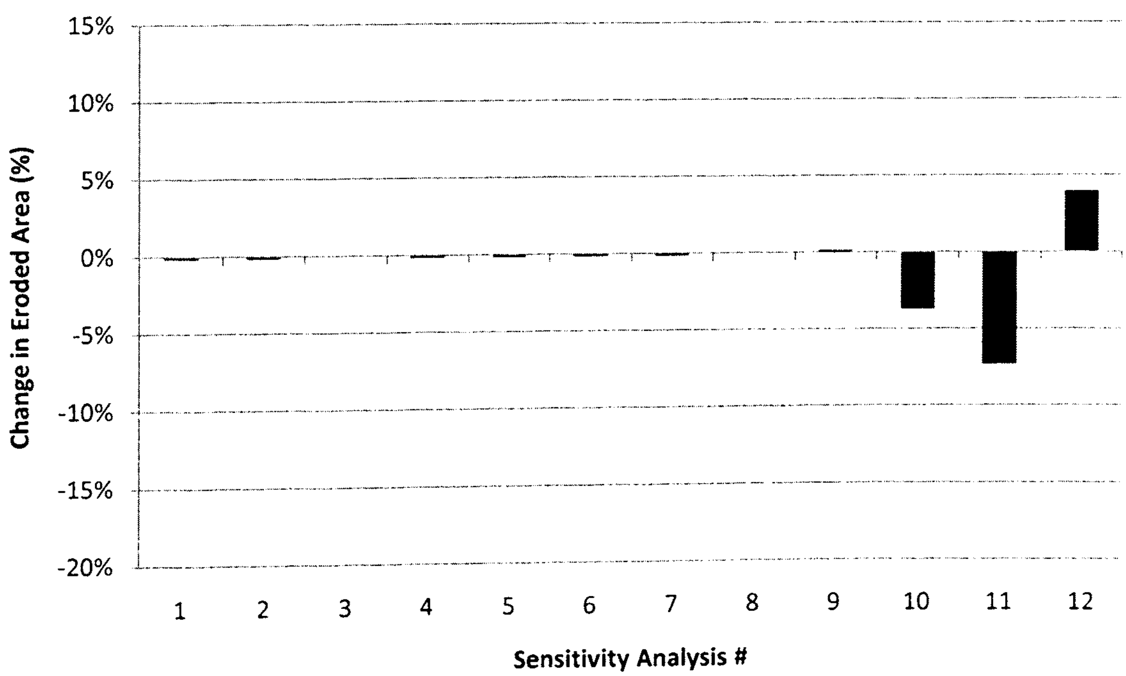

Figure 6.3 Percent Change in Eroded Area Varying Parameter Values by 10%

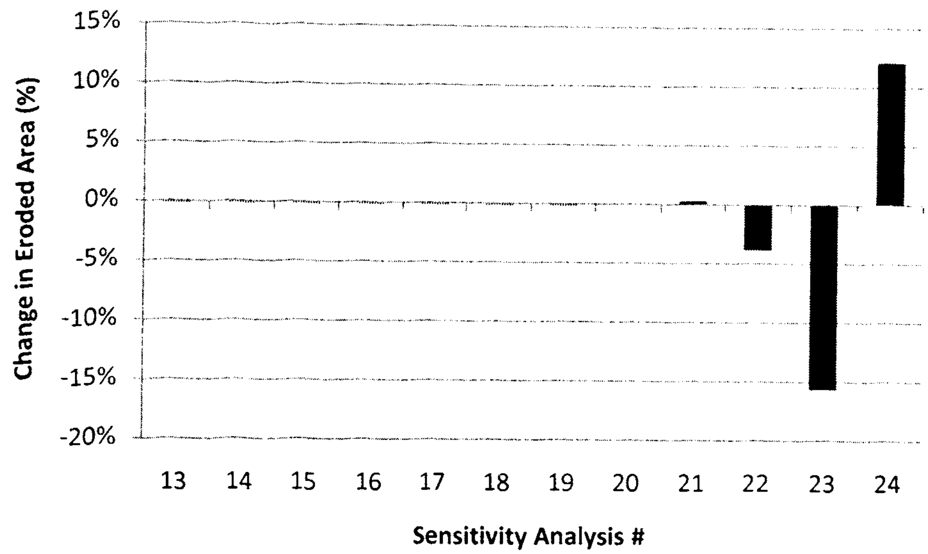

Figure 6.4 Percent Change in Eroded Area Varying Parameter Values by 20%

It is interesting to note that decreasing the critical shear stress of the soil caused only a very slight increase in the amount of erosion; however, an increase of the critical shear stress by 10% and 20% resulted in decreases of 3.7 and 3.8%, respectively, in the amount of area eroded from the cross-section with respect to the erosion occurring with the default parameters. The greater the critical shear stress of the soil, the less erosion of the cross-section occurred; however, there was only a slight difference between the erosion amounts that occurred with an increase of 10% versus 20%.

As anticipated, there is a positive correlation between the magnitude of the erodibility coefficient and the amount of eroded material. When the erodibility was decreased by 10 and 20%, the resulting erosion decreased by 7.3 and 15.6%, respectively. Conversely, an increase in the erodibility of 10 and 20% resulted in an increase in erosion of 3.9 and 12.0%, respectively.

In addition to comparing the area of eroded material and its relationship to variations in values of the soil parameters, the FSs at each time step in the hydrograph for the default values and the various sensitivity analyses were also compared. The percent decrease in the minimum FS and increase in the maximum FS were plotted for each analysis, and are shown in Figures 6.5 and 6.6. The parameter adjustments that caused

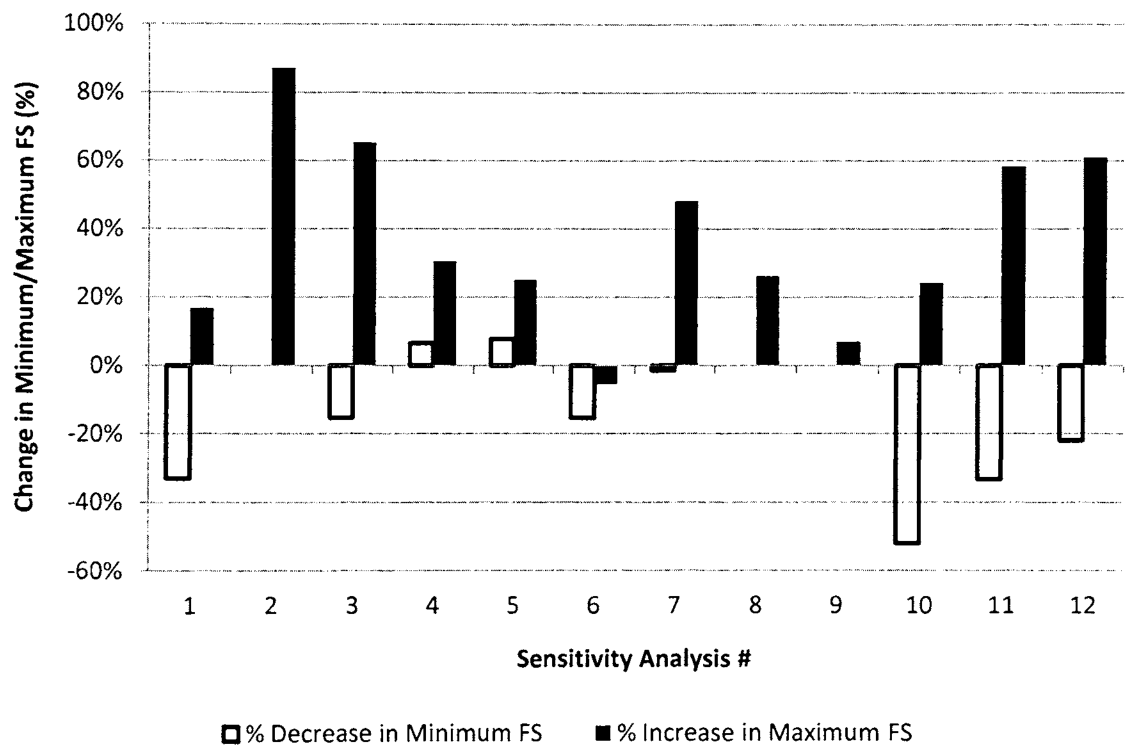

Figure 6.5 Percent Decrease in Minimum FS and Increase in Maximum FS (10%)

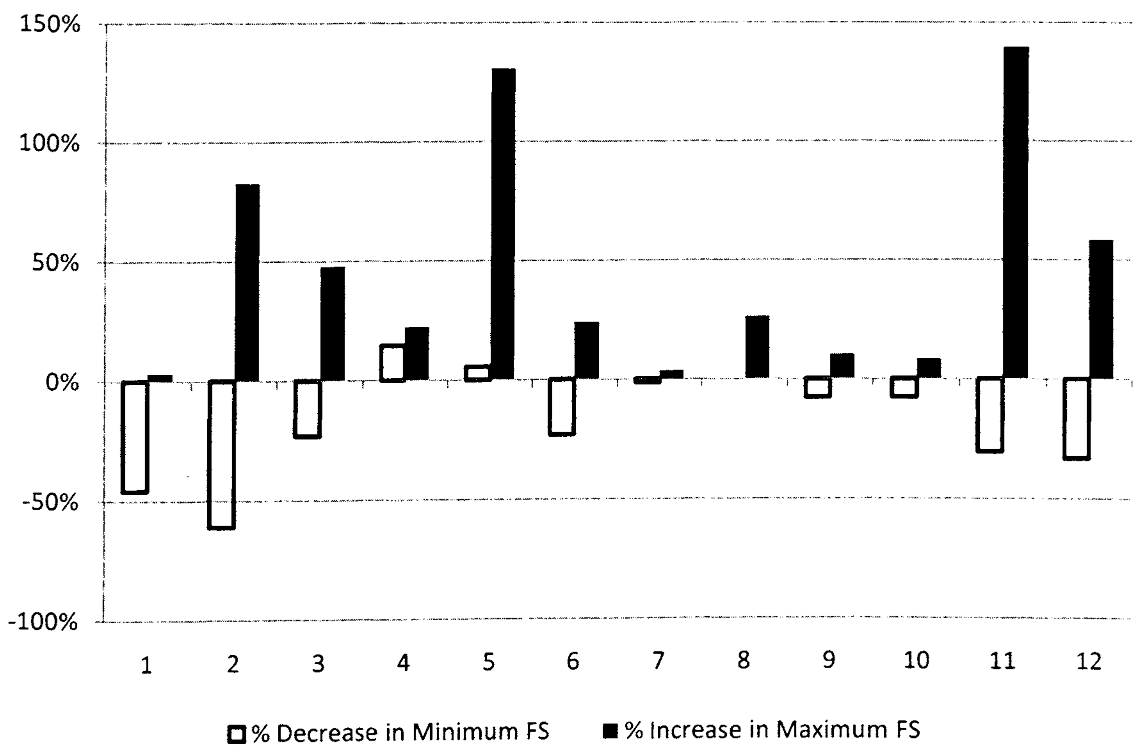

Figure 6.6 Percent Decrease in Minimum FS and Increase in Maximum FS (20%)

the greatest percentage range between the minimum and maximum FS were a decrease in the erodibility coefficient, an increase in the friction angle, an increase in the erodibility coefficient, a decrease in cohesion, and an increase in the critical shear stress for the soil, resulting in ranges of 91.6, 87.1, 82.9, 80.6 and 76.1%, respectively.

The FS at every time step was plotted for each given set of parameters, and all plots are provided in Appendix C. From these plots it was possible to determine those parameters which caused the greatest percent decrease and increase in the FSs for each scenario. An increase in the critical shear stress of the soil, decrease in the erodibility coefficient and decrease in the friction angle of the soils caused the greatest decreases in the minimum FS, with decreases of 52, 33.3 and 33%, respectively. An increase in the friction angle, decrease in cohesion, increase in erodibility coefficient and decrease in erodibility coefficient caused the greatest increases in the maximum FS, with increases of 87.1, 65.2, 60.9 and 58.3%, respectively.

Shortening the duration of the hydrograph also had a significant impact on the amount of erosion and the stability of the streambank. The total area of eroded material from the cross-section for the hydrographs where the time step was shortened from 24 hours to 12, 6 and 3 hours was 120.8, 141.6, and 147.2 m^2, respectively. This represents increases of 20.9, 41.7 and 47.3 % over erosion resulting from the original hydrograph with 24-hour time steps. These results indicate that the more quickly the water surface elevation rises and draws down within a cross-section, the more amount of material is eroded. This is an expected conclusion, as rapid changes in water surface elevation result in rapid changes in the velocity gradient and shear stresses within the channel, which are the predominant factors in determining the amount of erosion that occurs.

Shortening the time step of the outflow hydrograph did not have a significant impact on the magnitude of the minimum FS calculated for the cross-section. The minimum FS resulting from the original hydrograph was 0.16, and the minimum FSs from the hydrographs where the time step was shortened from 24 hours to 12, 6 and 3 hours were 0.06, 0.17 and 0.09, respectively. In all cases, the minimum FS occurred when the water surface elevation within the channel was also at its minimum, which is the portion of the hydrograph where confining pressure from the flow is at a minimum.

6.5. CONCLUSIONS

The results of the sensitivity analysis indicate that the soil parameters that have the greatest impact on both the amount of material eroded from the cross-section as well as the stability of the bank were the erodibility coefficient and the critical shear stress of the soil layers, indicating that the BSTEM analysis is most sensitive to changes in these parameters. The stability is also significantly affected by variations in the friction angle and cohesion of the soils. The findings of this sensitivity analysis are consistent with those of Jha et al. (2005), which determined that two of the four parameters to which bank erosion and stability are most sensitive are the erodibility coefficient and critical shear stress of the bank material [11].

The amount of material eroded is also affected by the rate at which the water surface elevations within the cross-section increase and decrease. More rapid changes in the flow elevation resulted in more material being eroded. Changes in the rate of the hydrograph did not, however, appear to impact the stability of the bank.

7. RESULTS AND DISCUSSION

The impacts of releases from Bagnell Dam were quantified by calculating the amount of eroded bank and toe material, the erosion rate, and the FS throughout various outflow hydrographs at each cross-section. The initial analyses were completed assuming that the phreatic surface was located at the ground surface throughout the entire duration of the hydrographs. Figures 7.1 through 7.22 illustrate the volume of eroded material per unit length along the reach (m^2) from the cross-sections during each outflow scenario, and the estimated average rate of erosion that occurred throughout each flood event.

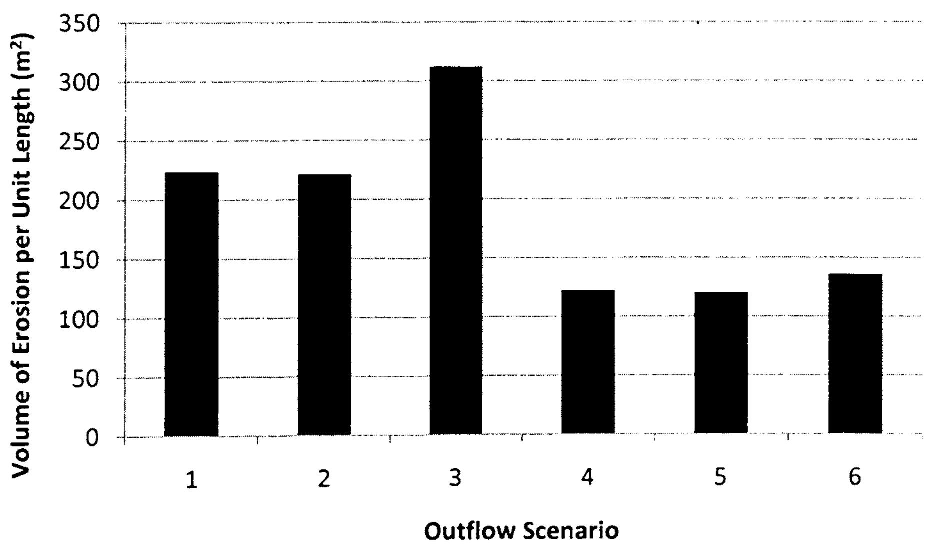

Figure 7.1 Cross-Section 1: Volume of Eroded Material per Unit Length

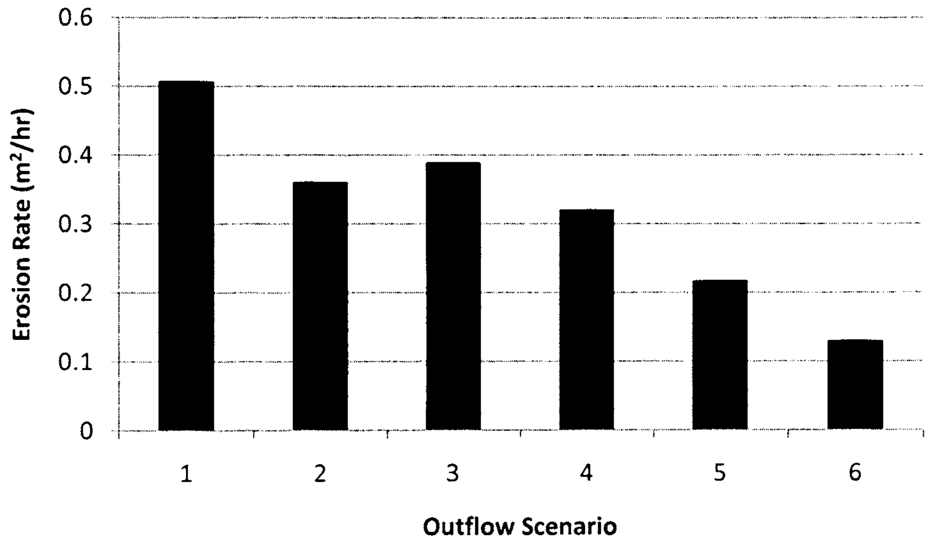

Figure 7.2 Cross-Section 1: Average Erosion Rate

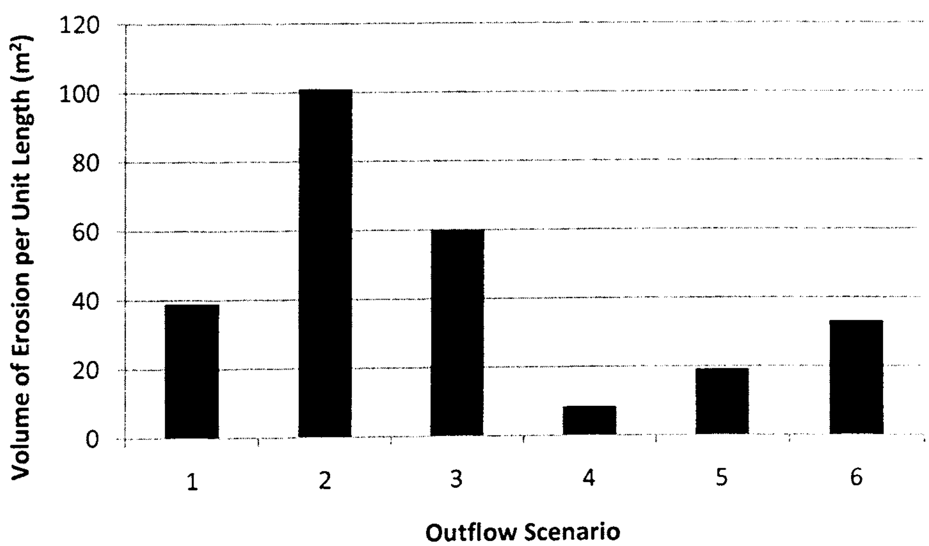

Figure 7.3 Cross-Section 2: Volume of Eroded Material per Unit Length

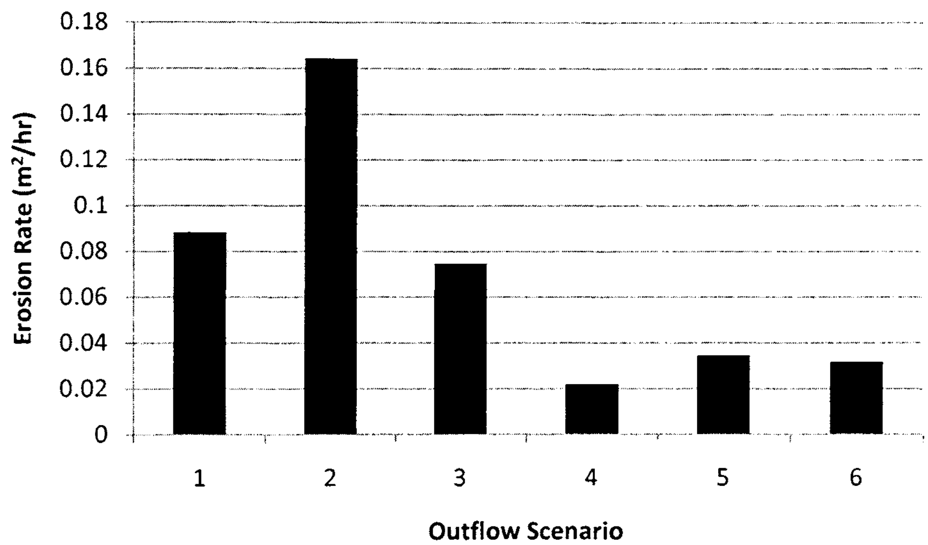

Figure 7.4 Cross-Section 2: Average Erosion Rate

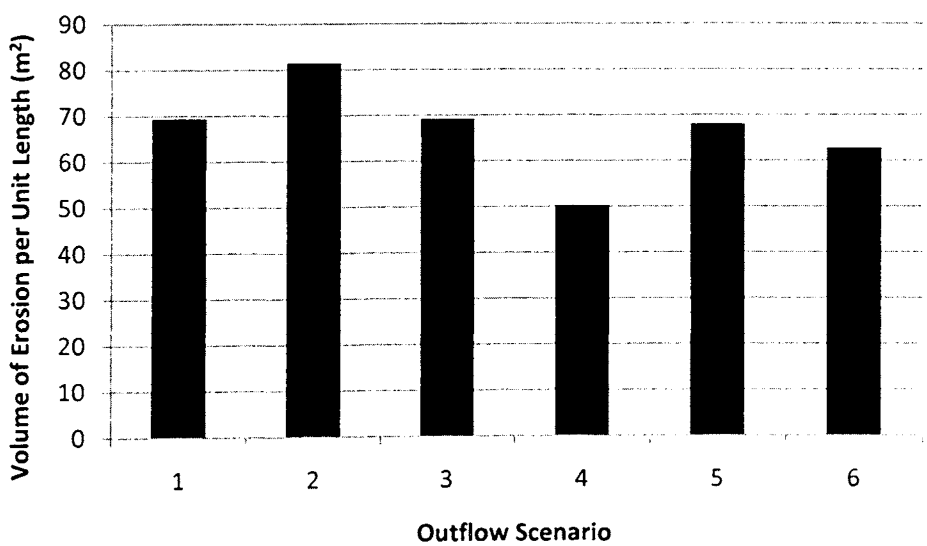

Figure 7.5 Cross-Section 3: Volume of Eroded Material per Unit Length

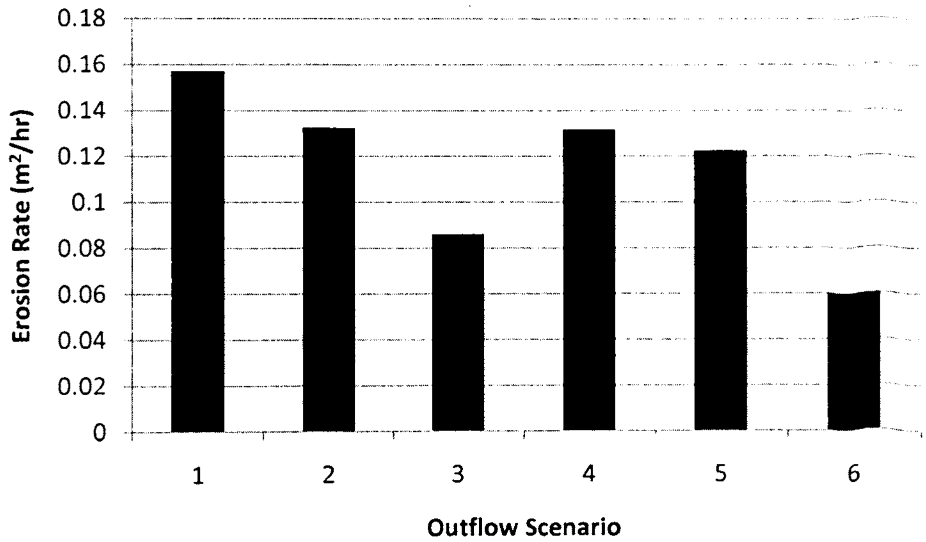

Figure 7.6 Cross-Section 3: Average Erosion Rate

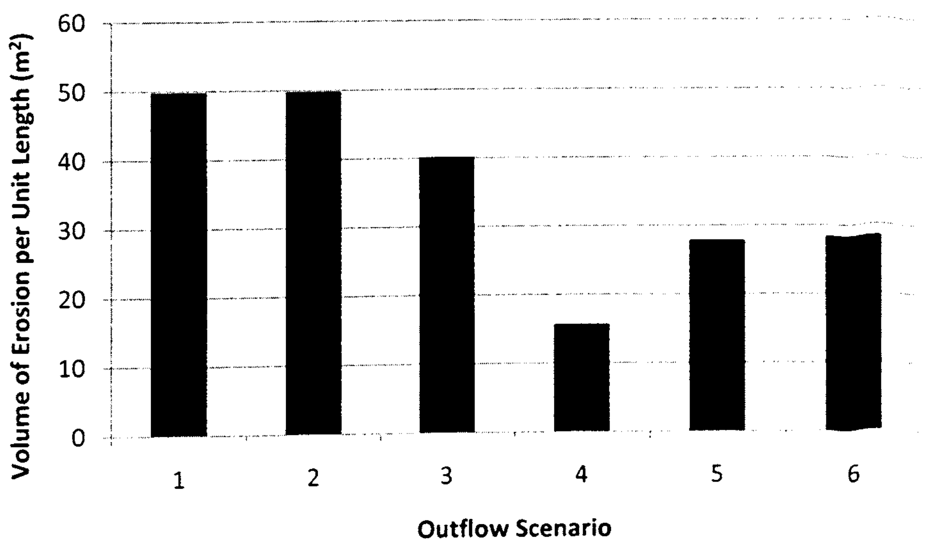

Figure 7.7 Cross-Section 4: Volume of Eroded Material per Unit Length

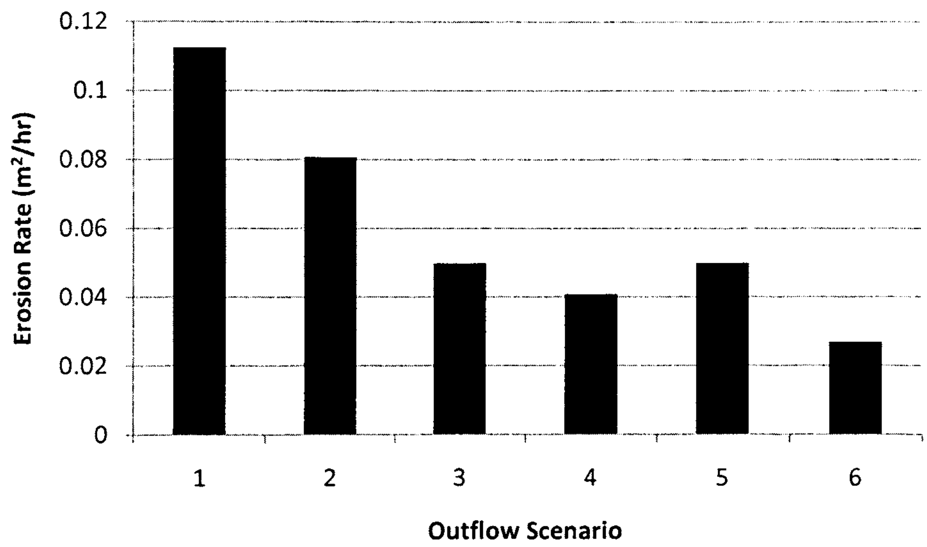

Figure 7.8 Cross-Section 4: Average Erosion Rate

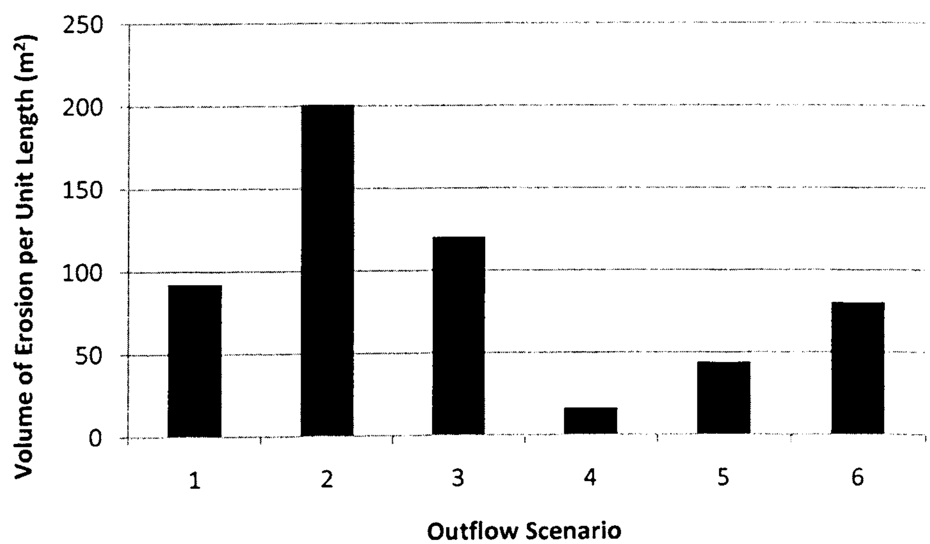

Figure 7.9 Cross-Section 5: Volume of Eroded Material per Unit Length

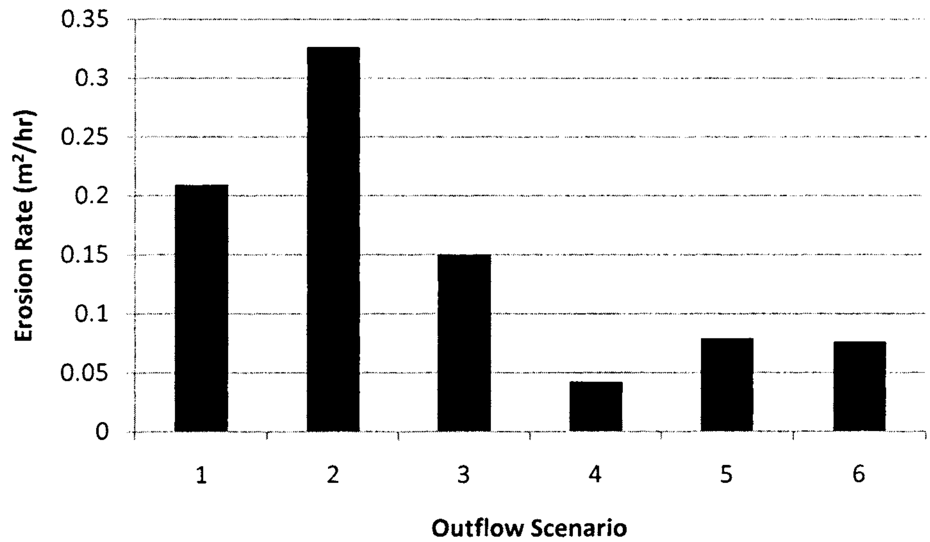

Figure 7.10 Cross-Section 5: Average Erosion Rate

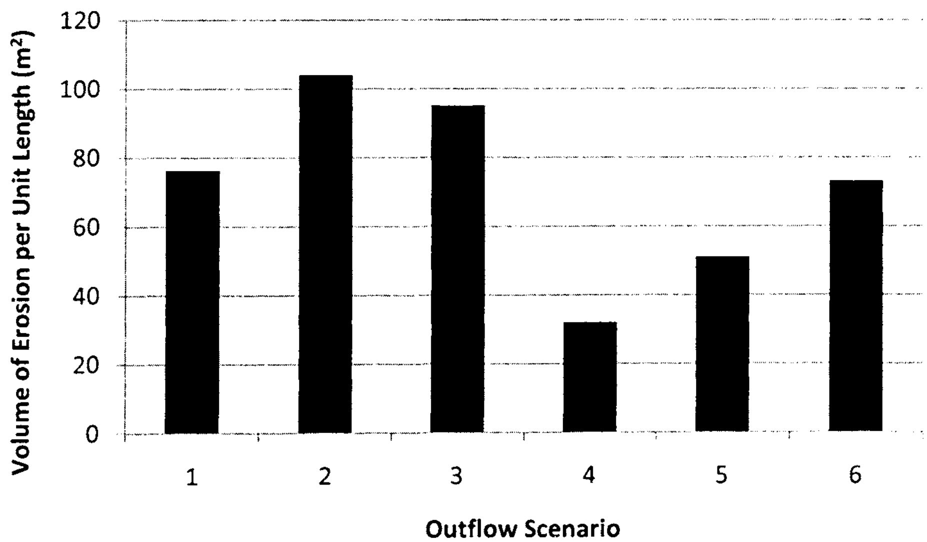

Figure 7.11 Cross-Section 6: Volume of Eroded Material per Unit Length

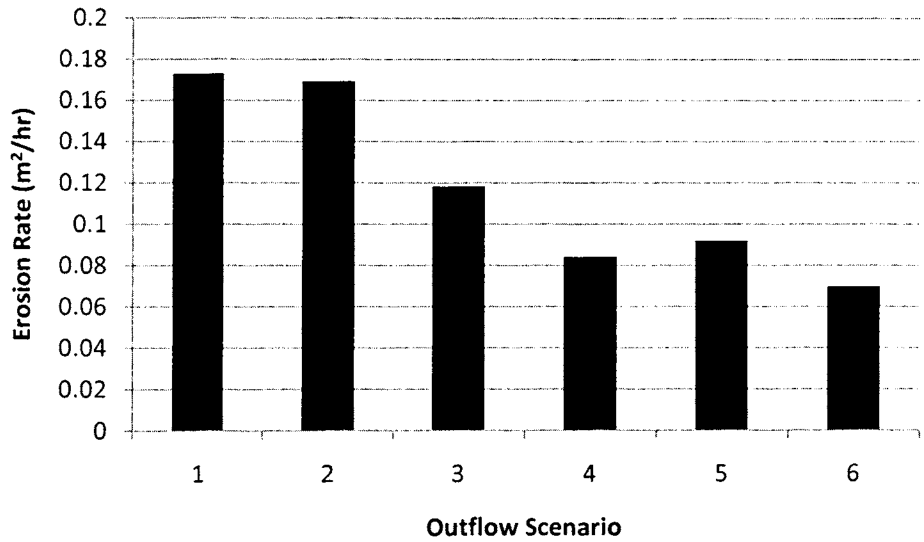

Figure 7.12 Cross-Section 6: Average Erosion Rate

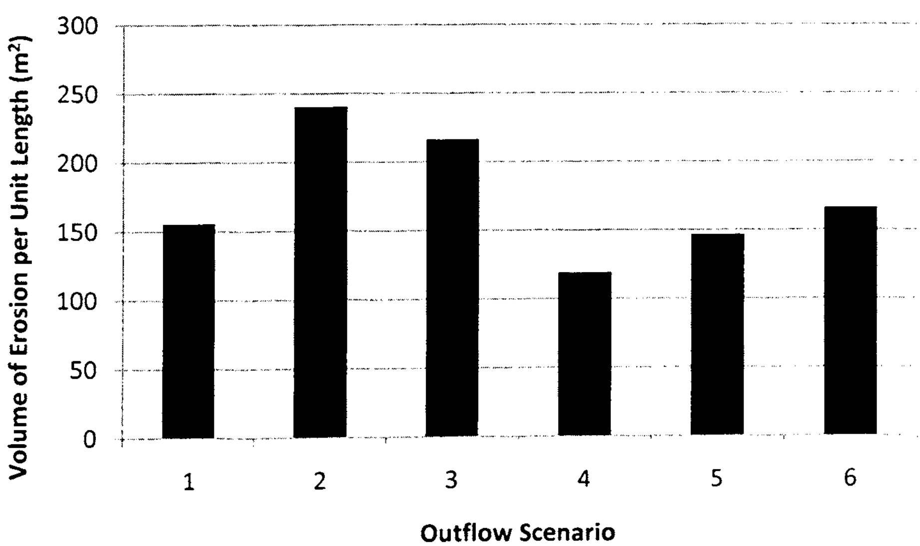

Figure 7.13 Cross-Section 8: Volume of Eroded Material per Unit Length

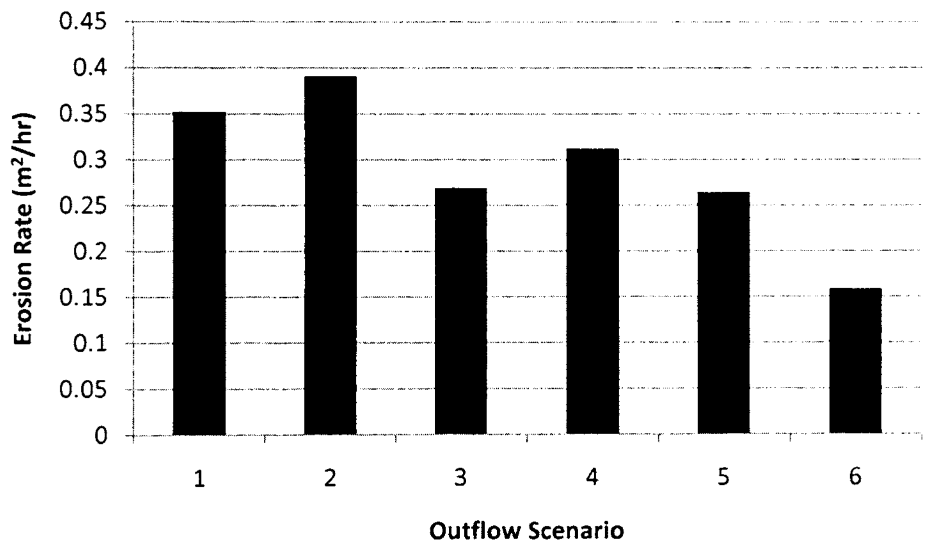

Figure 7.14 Cross-Section 8: Average Erosion Rate

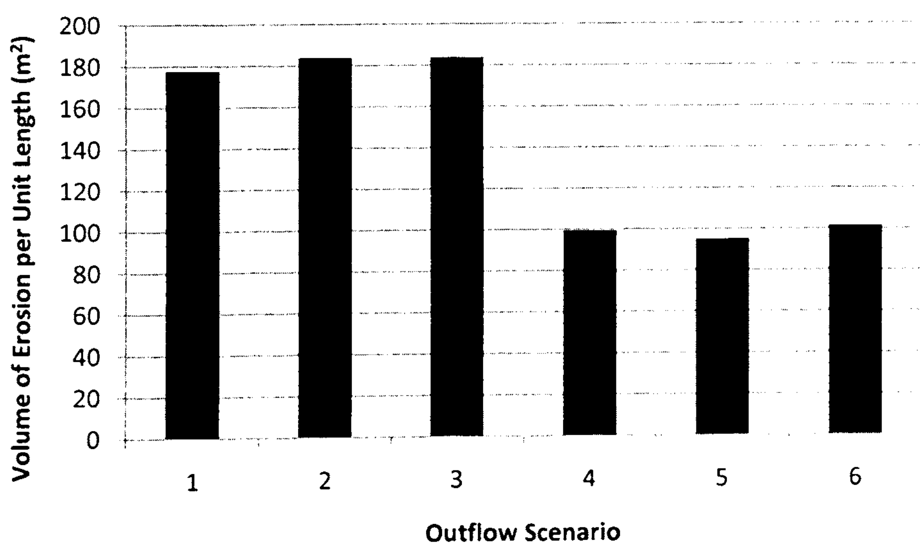

Figure 7.15 Cross-Section 9: Volume of Eroded Material per Unit Length

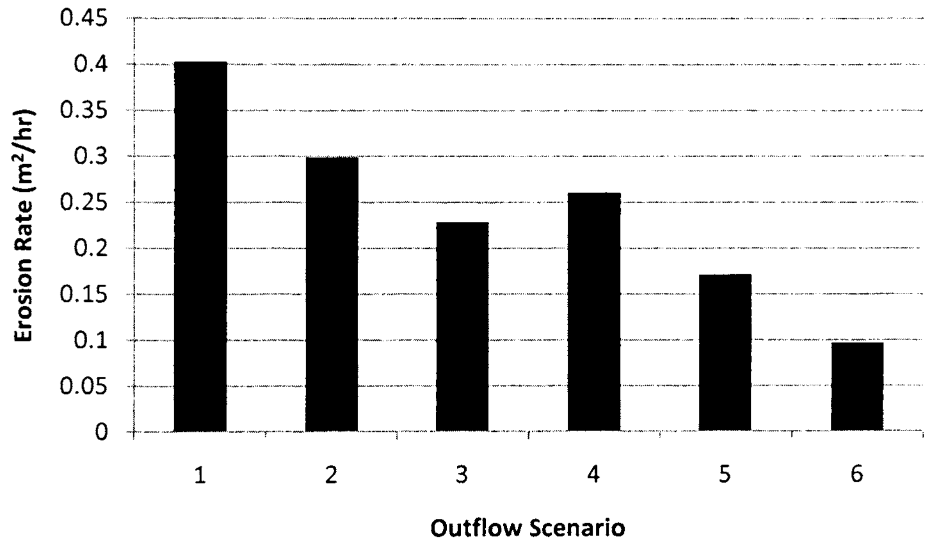

Figure 7.16 Cross-Section 9: Average Erosion Rate

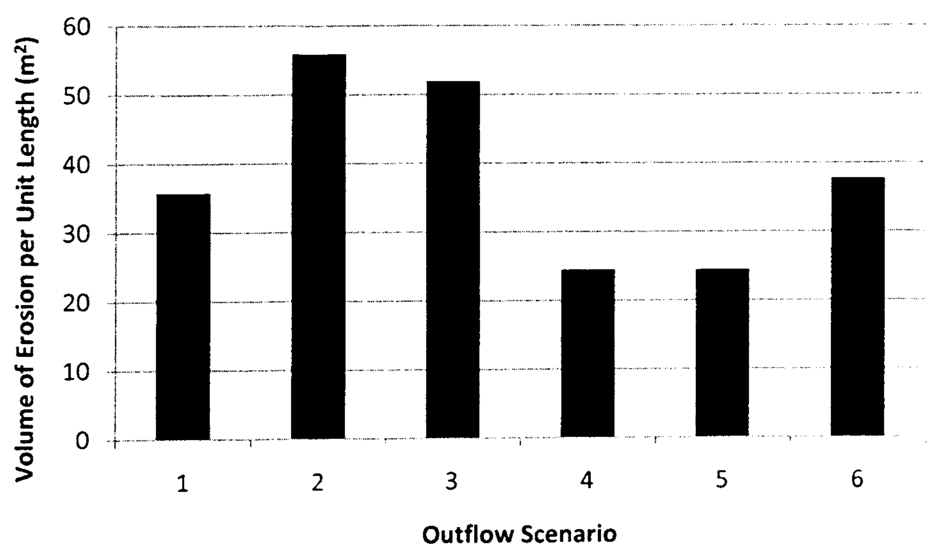

Figure 7.17 Cross-Section 10: Volume of Eroded Material per Unit Length

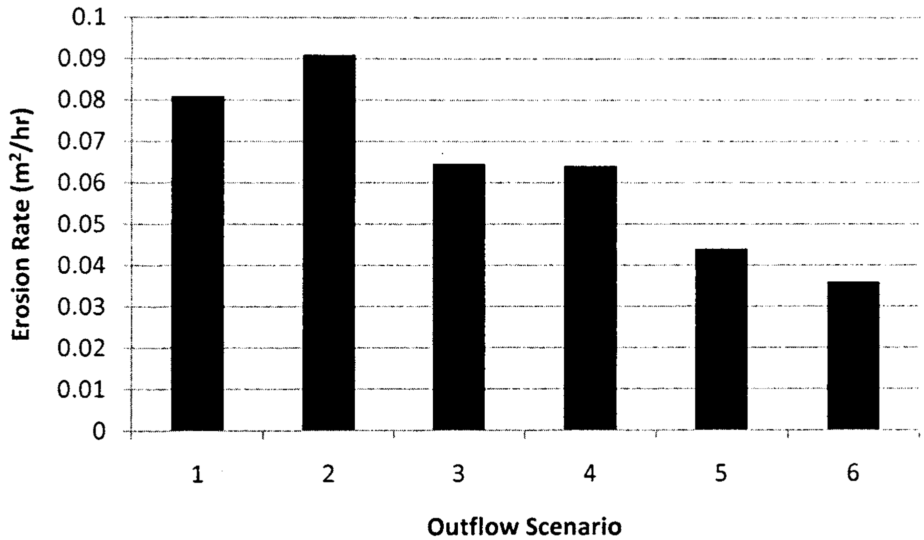

Figure 7.18 Cross-Section 10: Average Erosion Rate

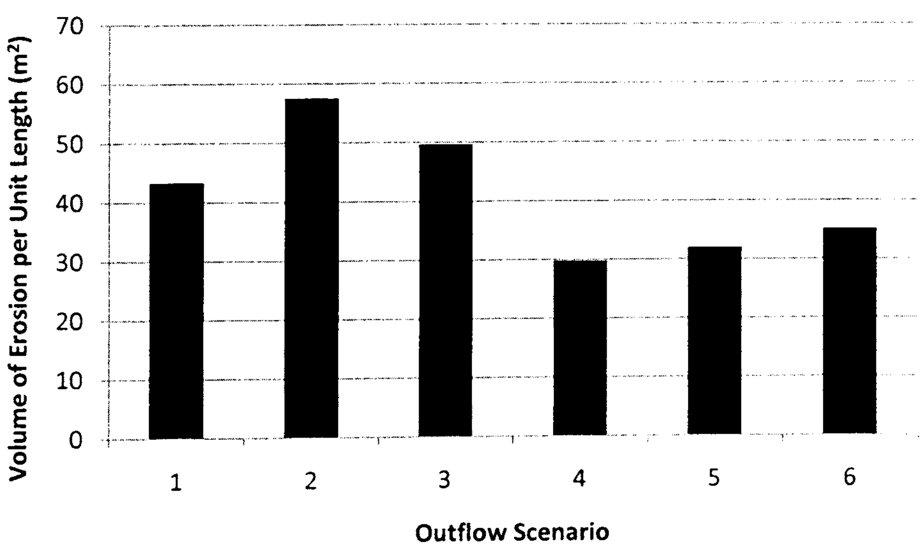

Figure 7.19 Cross-Section 11: Volume of Eroded Material per Unit Length

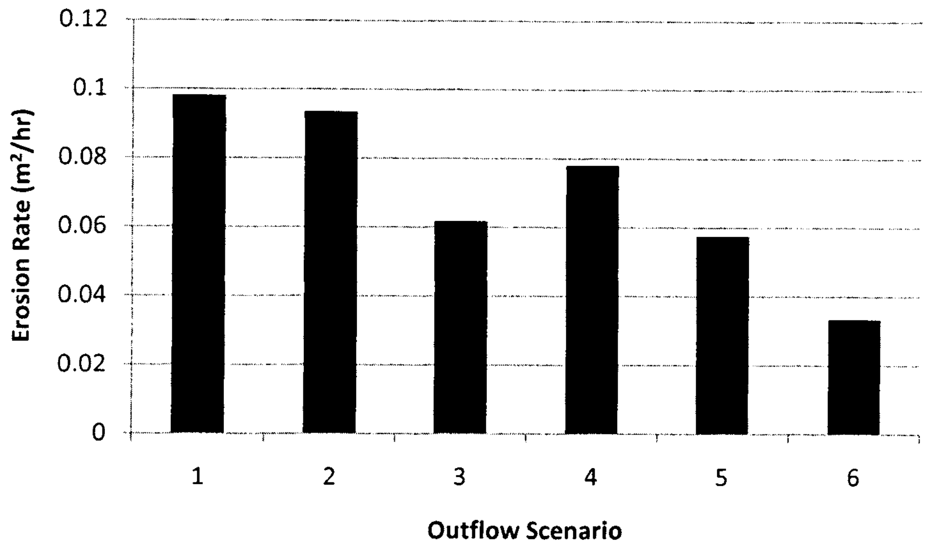

Figure 7.20 Cross-Section 11: Average Erosion Rate

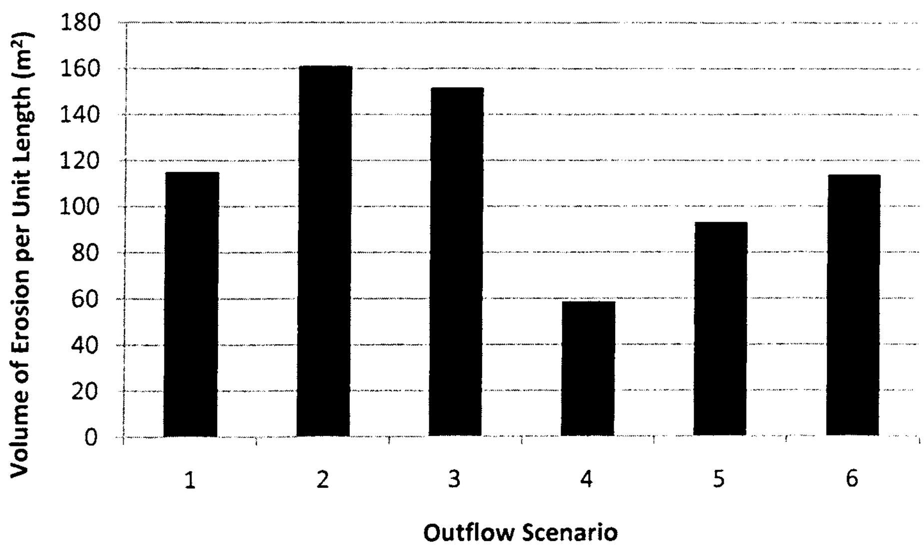

Figure 7.21 Cross-Section 14: Volume of Eroded Material per Unit Length

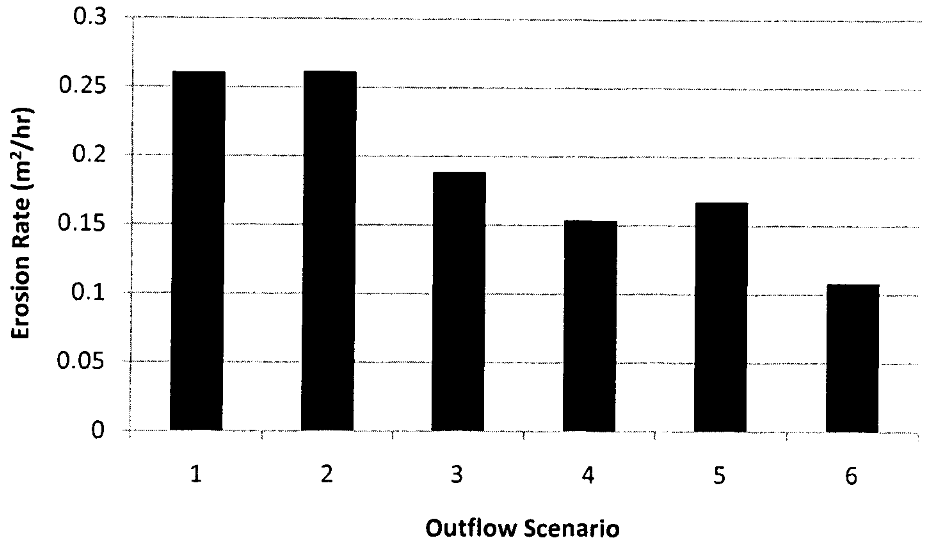

Figure 7.22 Cross-Section 14: Average Erosion Rate

These graphs allow for a visual comparison of the effects of the various outflow scenarios. The High Flow scenario caused the greatest amount of erosion for 10 of the 11 cross-sections. The High Flow, Summer Generation and Hypolimnia scenarios caused the greatest amount of erosion in 6 cross-sections, in order from highest to lowest. For 8 of the cross-sections, the Summer Low Flow scenario caused the least amount of erosion.

The Hypolimnia scenario caused the highest rate of erosion (m^2/hr) in 6 of the cross-sections, and the erosion rates of the remaining 5 cross-sections were highest during the High Flow scenario. In 6 of the cross-sections, the 3 outflow scenarios that caused the highest erosion rates were Hypolimnia, High Flow and Summer Generation, in order from highest to lowest. The Fall Rebound scenario caused the lowest erosion rate in 9 of the 11 cross-sections.

Based on these findings, it was determined that the Hypolimnia, High Flow and Summer Generation scenarios were the most critical for the Osage River downstream of Bagnell Dam. A stability analysis for these 3 scenarios was performed for all cross-sections. For 8 cross-sections, the Summer Generation scenario resulted in the lowest FS or highest bank instability. One of the cross-sections, cross-section 3, was most unstable during the High Flow scenario, and the remaining 2 sections, numbers 6 and 9, were stable throughout the entire duration of all three scenarios. Figures 7.23 through 7.26 are plots depicting the variations of the FS for several cross-sections during the most critical scenario for each cross-section. The water surface elevation is also plotted alongside the FS to illustrate the impacts of the fluctuating water surface elevation on the FS. Appendix D contains the remaining plots of the critical outflow scenarios for those cross-sections not included below. Cross-sections 6 and 9 are not included in this Appendix because they did not fail in BSTEM for any of the outflow scenarios.

In terms of the total amount bank and toe erosion, the most critical outflow scenario was the High Flow scenario. The second and third most critical scenarios were the Summer Generation and Hypolimnia scenarios, respectively. This is consistent with the findings of the 2003 erosion analysis. When evaluating the average erosion rates, the most critical scenarios are Hypolimnia, High Flow and Summer Generation, in order from highest to lowest erosion rates. The Summer Generation scenario was the most critical in terms of bank instability along the lower Osage River, followed by the High Flow scenario. An example of a typical failure surface is provided in Figure 7.27. This figure also demonstrates the changes that occurred due to bank and toe erosion.

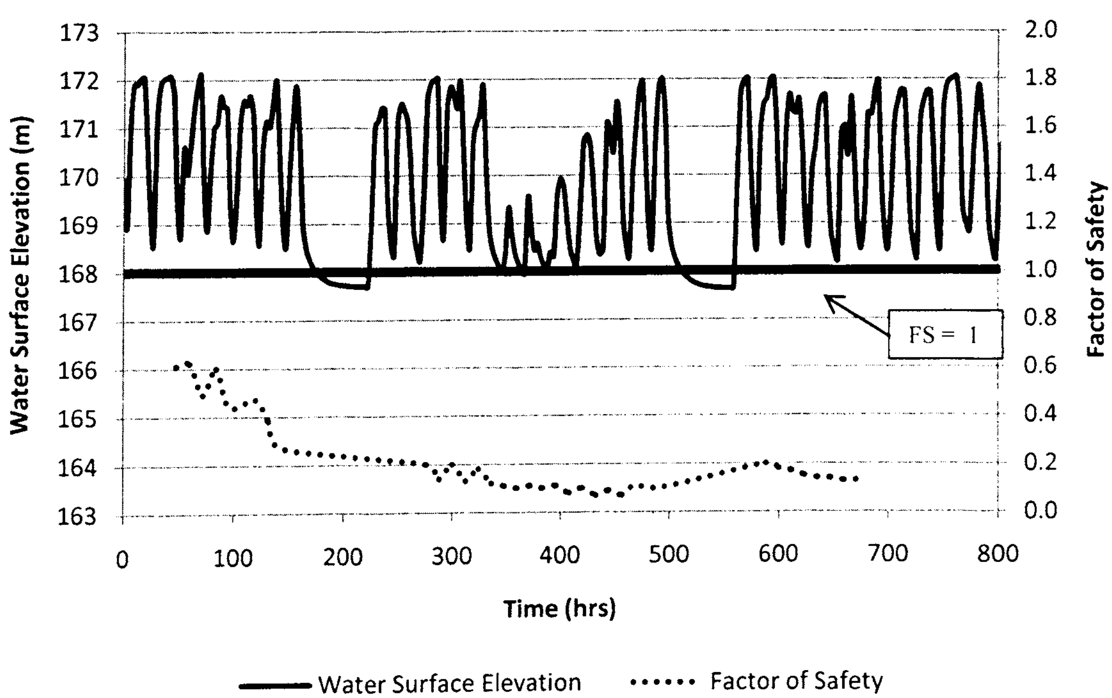

Figure 7.23 Summer Generation Scenario at Cross-Section 1

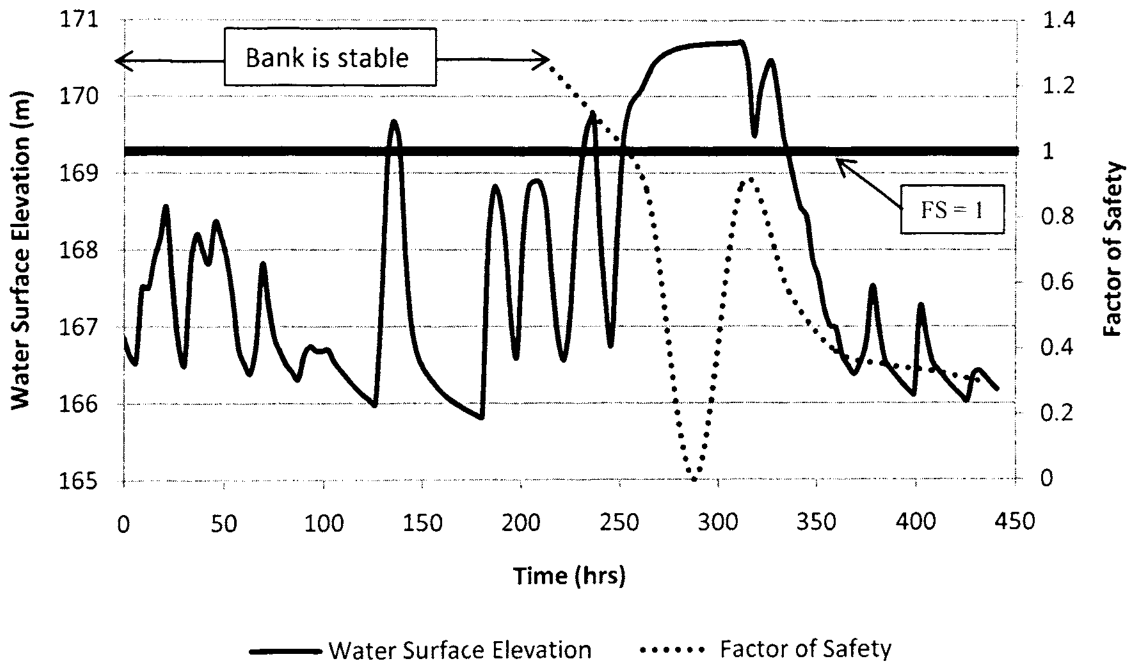

Figure 7.24 High Flow Scenario at Cross-Section 3

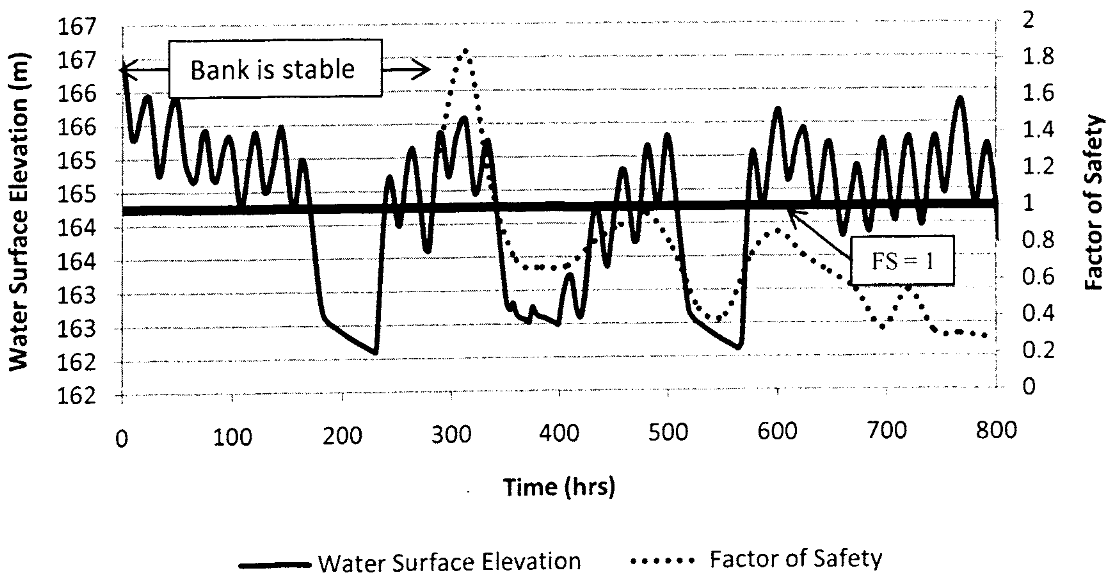

Figure 7.25 Summer Generation Scenario at Cross-Section 8

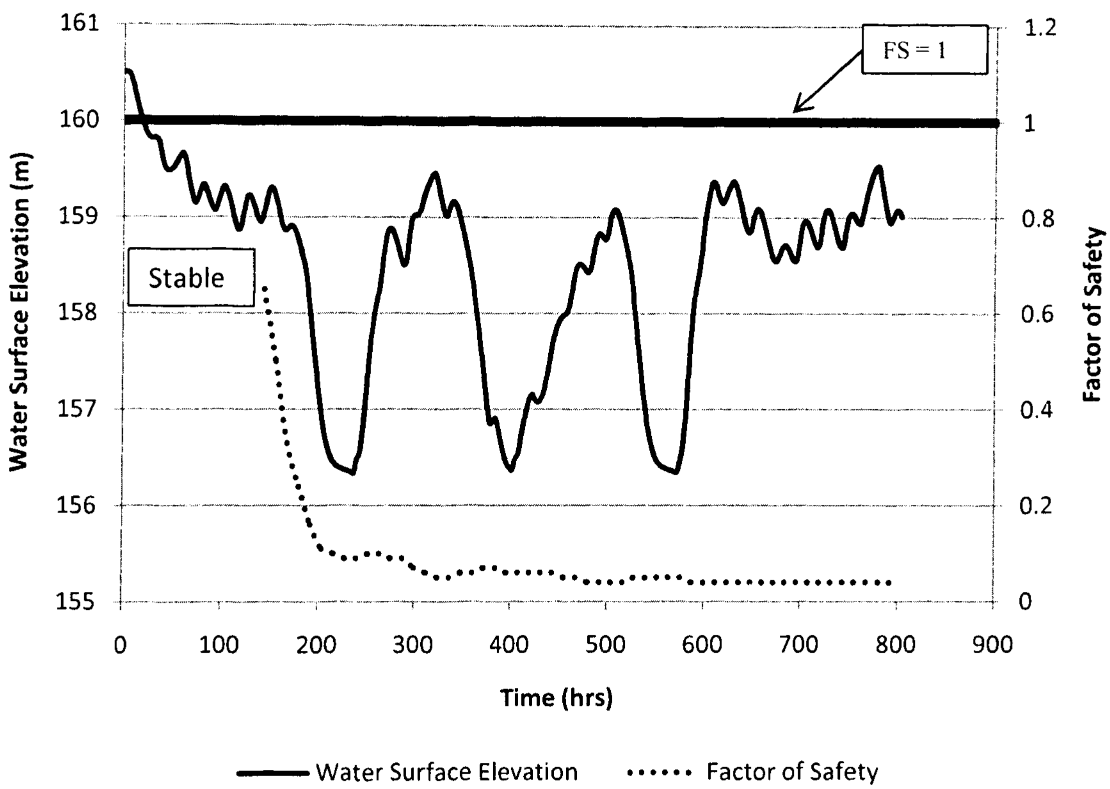

Figure 7.26 Summer Generation Scenario at Cross-Section 14

A stability analysis was also completed for all the cross-sections neglecting the effects of bank toe erosion. During this analysis, the minimum FS always occurred when the water surface elevation within the channel was at a minimum. Without taking into account toe erosion, 7 of the 11 cross-sections were either conditionally stable or stable for all ranges of water surface elevations evaluated. This suggests that the banks along the lower Osage River are actually quite stable during all of the outflow scenarios when toe erosion is omitted from the analysis. This finding is consistent with Simon et al. (2003), in which it was determined that the banks of the Missouri River were very stable when non-eroded; however, when the analysis accounts for toe erosion, bank failure becomes increasingly more likely [9].

The effects of varying the assumption regarding depth to the phreatic surface are shown in Figures 7.28 through 7.30. GW Assumption 1 is the original assumption that the phreatic surface remained at the ground surface, and GW Assumption 2 assumed a

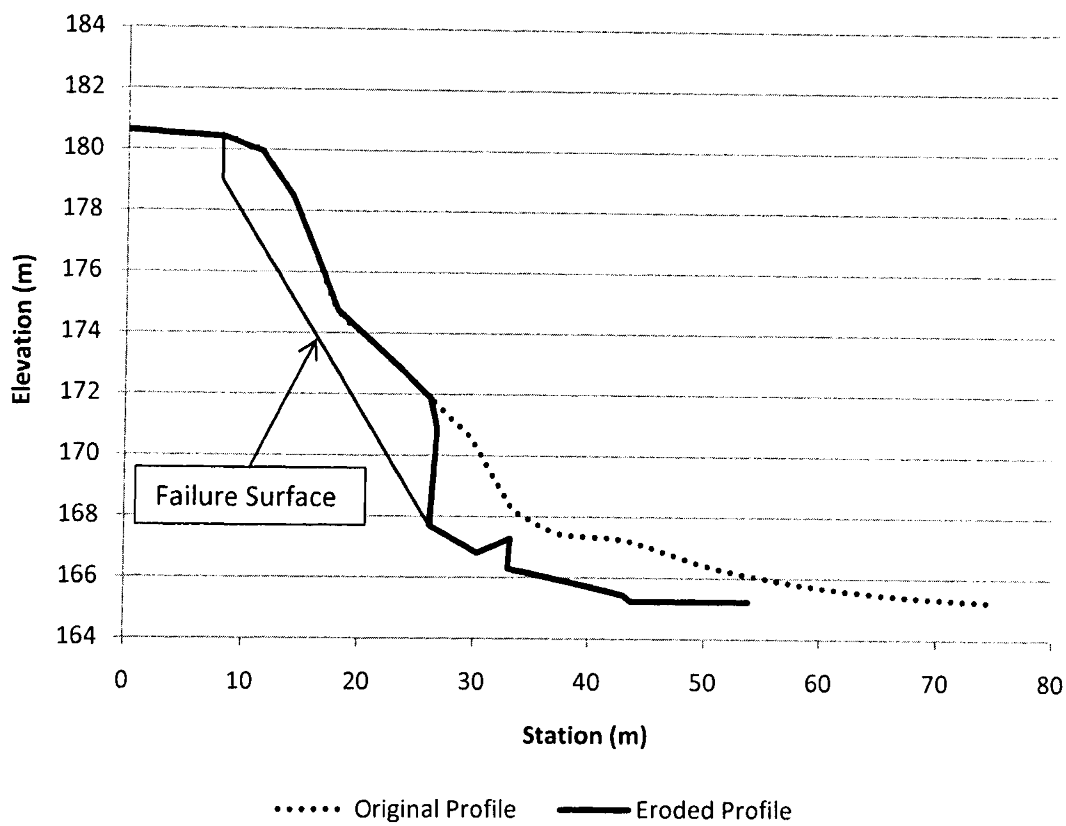

Figure 7.27 Summer Generation Scenario at Cross-Section 2: Failure Surface

drawdown of the phreatic surface equal to one-tenth of drawdown of the water surface elevation. As demonstrated by these plots, when the water table within the bank was assumed to drawdown at a rate equal to one-tenth the drawdown rate of the water surface elevation, the bank was significantly more stable throughout the studied outflow scenarios. The FS at some time steps within these scenarios was greater than 4 times the FS determined when assuming that the phreatic surface remained at the ground surface. This vast difference in stability points out the dependency of the bank stability on the water table elevation within the bank, and the resulting pore water pressures. This is also consistent with Simon et al. (2003), which found that infiltration, toe erosion and geotechnical failure all work together to promote bank instability, and the omission of even one of these factors can lead to the stability to be significantly overestimated [9].

This report also recommended the use of a seepage model coupled with the stability model in order to obtain the most accurate results.

The effects of the hydrograph drawdown rate were discussed at length in the sensitivity analysis. The results of varying the rate of rise and drawdown of the hydrographs indicated that increasing the drawdown rate of the water surface elevation resulted in more erosion of the bank and toe, but did not affect bank stability significantly. This is inconsistent with the results reported in Simon et al. (2003), which indicated that a slower drawdown allows for phreatic surface to drawdown sufficiently such that the pore water pressures dissipate, and the bank is more stable [9]. The source of this inconsistency is that the phreatic surface in the current study was assumed to remain at the ground surface throughout this portion of the analysis, and the dissipation of pore water pressures was not taken into account. It is likely that if the pore water pressures were decreased in this portion of the study, the stability analysis would demonstrate that the banks are indeed more stable with a slower drawdown rate.

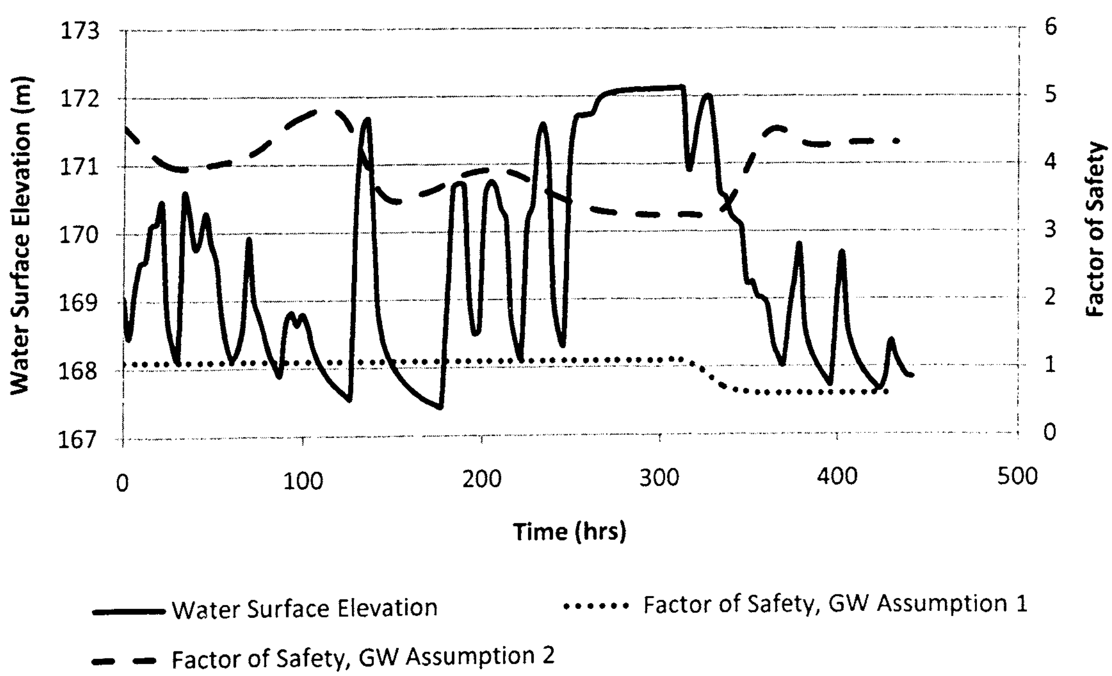

Figure 7.28 Hypolimnia Scenario at Cross-Section 2 Varying GW Assumptions

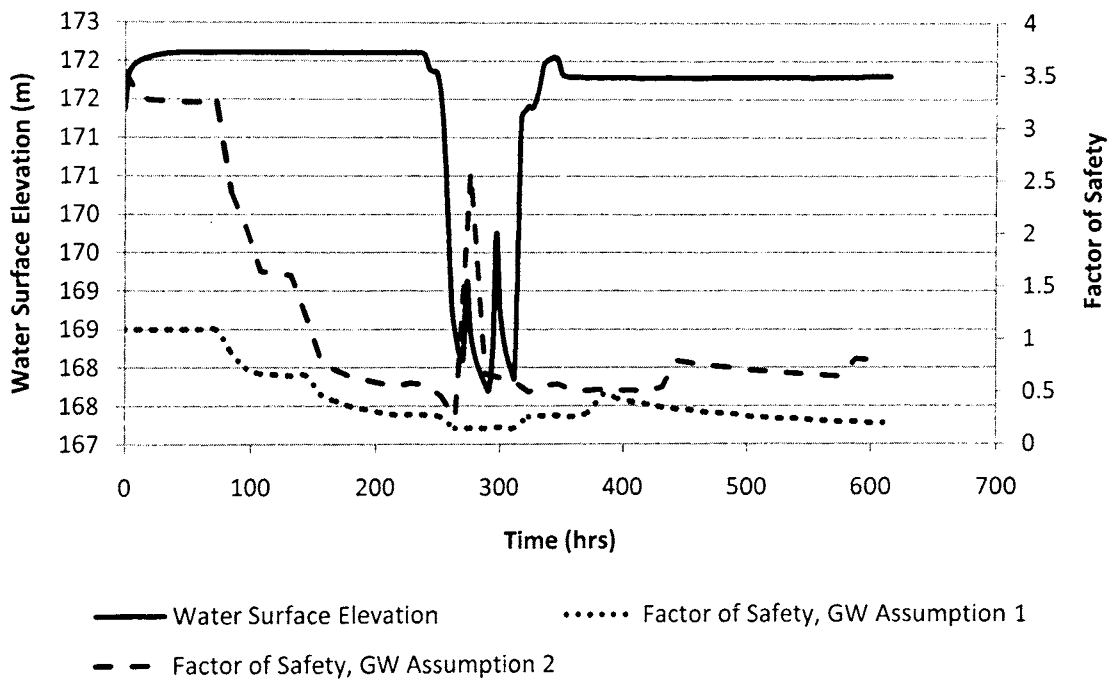

Figure 7.29 High Flow Scenario 2 at Cross-Section 2 Varying GW Assumptions

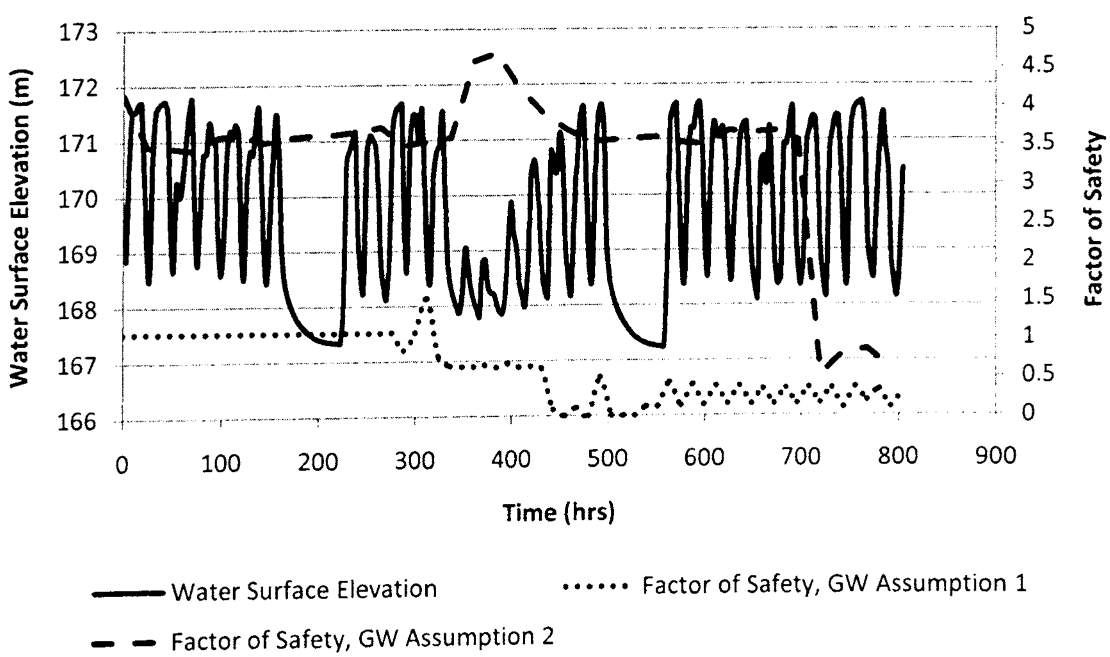

Figure 7.30 Summer Generation Scenario at Cross-Section 2 Varying GW Assumptions

Comparison plots of the original cross-section geometry and the most critical eroded cross-section geometry for all sections are provided in Appendix E. The most critical eroded geometries are those resulting from the outflow scenario that caused the greatest amount of erosion. In addition, plots showing the fluctuations of the FS during various outflow scenarios are provided in Appendix D.

8. CONCLUSIONS AND RECOMMENDATIONS

The results of this analysis indicate that the outflow scenarios resulting in the greatest amount of bank and toe erosion were the High Flow, Summer Generation and Hypolimnia scenarios, in order from the greatest amount of erosion to the least. The Summer Generation scenario was the most critical in terms of bank instability along the lower Osage River, followed by the High Flow scenario.

The overall stability of the banks is largely dependent on the depth of the phreatic surface and whether bank and toe erosion were included in the analysis. The stratigraphy of the banks contributed greatly to the failure mechanism that was most common in this analysis. The majority of the cross-sections were composed of cohesive soils in the upper portion of the bank supported by noncohesive soils. This resulted in the prevailing failure mechanism being mass wasting or cantilever failures that occurred when the toe had eroded to the point where there was not sufficient support for the cohesive layer.

The rate at which the hydrographs ramped up and drew down also impacted the stability of the banks. While the rapid changes in outflow did not contribute directly to bank instability, they resulted in more erosion of the bank toes, which subsequently led to mass wasting and cantilever failures. It is possible that slowing the ramp up and drawdown phases of the critical outflow scenarios would result in less toe erosion and less subsequent mass wasting, but this may not be possible due to the need to meet peaking demands. If possible, it may be beneficial to attempt to determine typical peak power demands and when they occur, so that releases may begin being made before the demands are at their peaks. This would allow for a slower ramp up phase of the outflow hydrographs. In addition, once the peak power demands have passed, slowing the rate at which releases are decreased would result in less erosion and possibly less bank instability. Slowing the drawdown phase would also allow for pore water pressures to decrease while at the same time providing for continued confining pressure from the river.

This analysis provided quantitative data demonstrating the effects of the various outflow scenarios from Bagnell Dam on erosion and stability of the banks of the lower Osage River. Although the data used as the basis for this analysis was reasonably accurate, a more refined model could be developed if additional, more detailed field data were collected. Based on the results of the sensitivity analysis and the impacts of varying the water table elevation, it is possible to recommend the most useful additional data to collect. It is important to narrow the data collection to only those variables that will have the greatest impact on the outcome of the analysis, because there is a large amount of geotechnical and groundwater data that can be collected, and this can be expensive and time-consuming.

The effects of varying the water table elevation have a significant effect on bank stability. Because it is very difficult to accurately predict the drawdown rate of the water table within a bank in relation to the drawdown of the flow within the channel, obtaining accurate groundwater data and its relationship to variations in the water surface elevation would improve significantly the accuracy of the stability analysis of the lower Osage River. This could be accomplished through the installation of piezometers in the streambanks at each cross-section location, and the continuous monitoring of water table elevations and streamflow elevations.

The accuracy of the stability analysis could also be increased by incorporating a seepage analysis into the stability analysis. If possible, one or more programs could be used so that fluvial erosion, pore water pressures, stability, and how they affect one another could all be analyzed simultaneously. This is consistent with Darby et al. (2007) which recommended the "coupling [of] a hydraulic erosion model with a finite element seepage analysis and limit equilibrium stability methods to address transient mass wasting triggered by bank profile deformation and/or variations in bank pore water pressures" [8].

In addition to the collection of groundwater data, there are several geotechnical properties of the streambank soils that could be measured to enhance the confidence in the accuracy of the erosion and stability analysis. The two most significant ones are the critical shear stress of the soil and its erodibility. If it is possible to collect data in addition to these properties, the friction angle and cohesion of the various soils would be the properties with the next highest priority in terms of impact on stability.

It may be possible to reduce the adverse effects of hydropower releases from Bagnell Dam on bank erosion and instability through the installation of toe protection along the lower Osage River. If the amount of toe erosion could be significantly reduced, the stability of the banks would be greatly increased, resulting in fewer mass failures and less bank migration. BSTEM contains an option to include toe protection in the erosion analysis. Additional model runs which include toe protection can then be carried out to determine the best form of protection, if any, for each stream reach.

This study provided a thorough analysis of the impacts of various outflow release patterns from Bagnell Dam on erosion and bank stability along the lower Osage River. This is especially important to landowners whose property abuts the river, as migration of the banks results in a loss of property. The most critical outflow scenarios were identified, as well as recommendations for future refinements of the model to possibly introduce optimum adjustments to the outflow patterns in order to minimize the detrimental effects to the streambanks.

APPENDIX A

MAP SHOWING CROSS-SECTION LOCATIONS

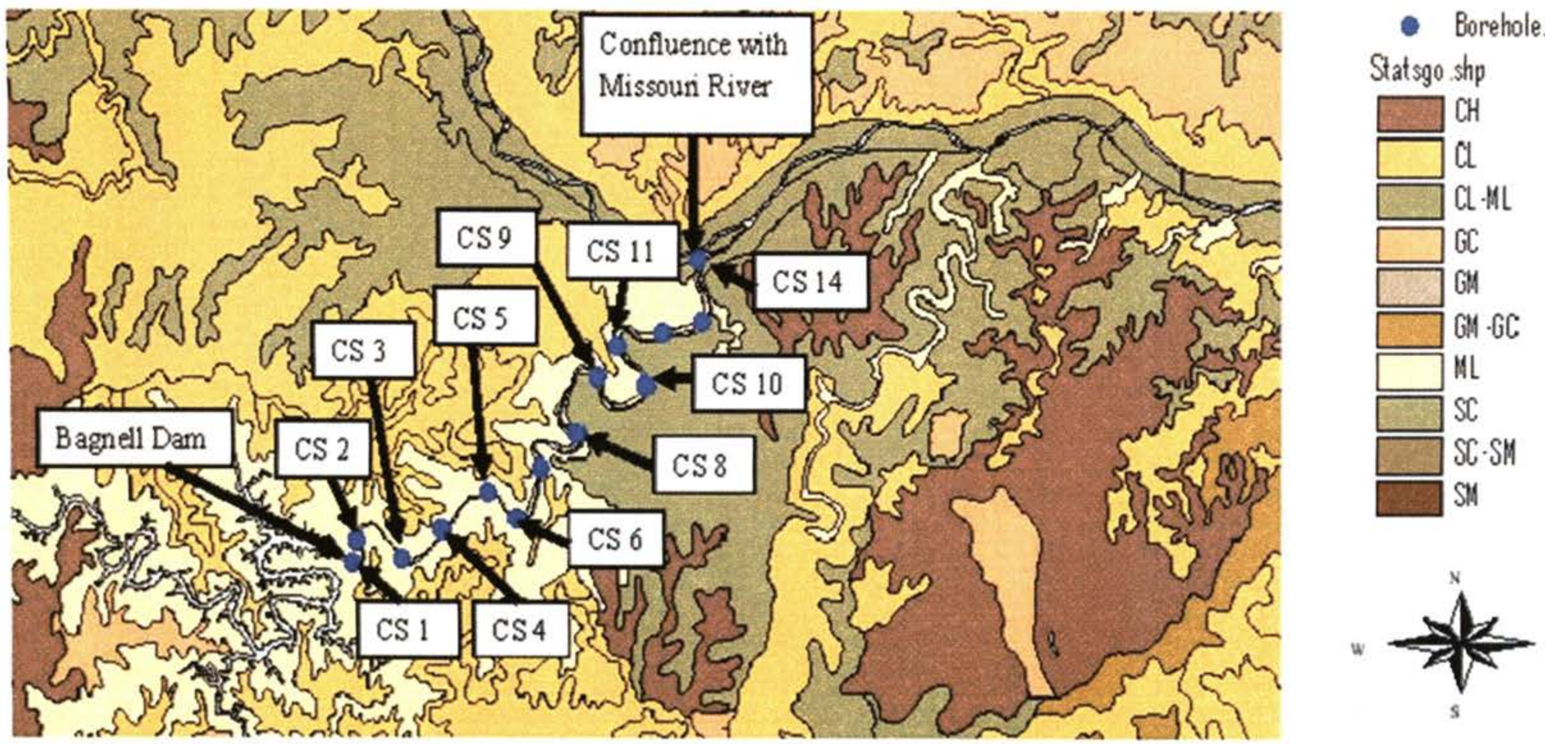

Figure A.1 Map Showing Cross-Section Locations [2]

APPENDIX B

CROSS-SECTION DATA

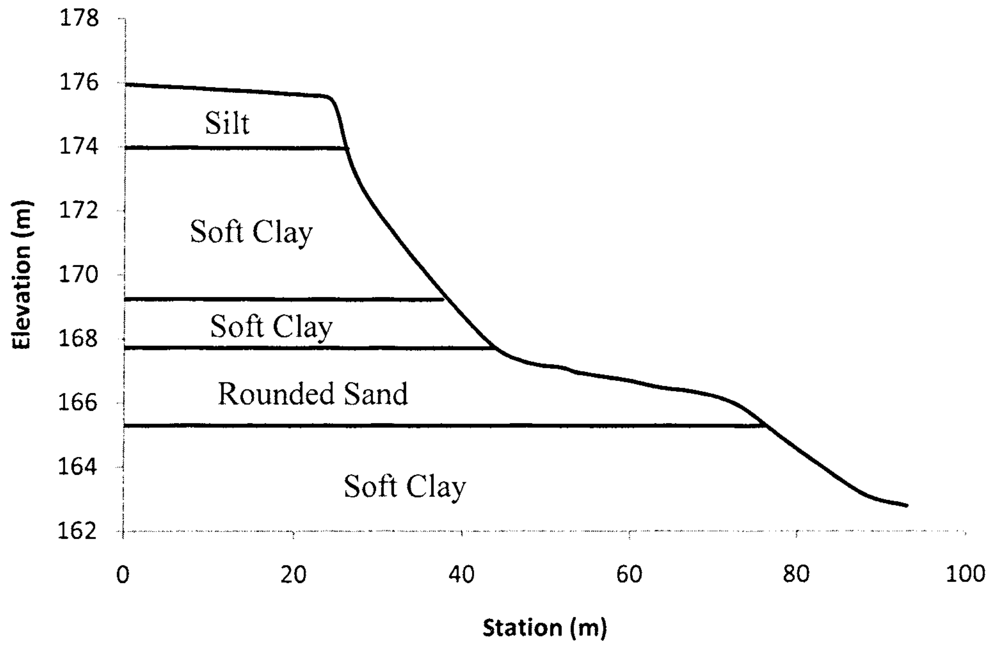

Figure B.1 Cross-Section 1 Geometry and Stratigraphy

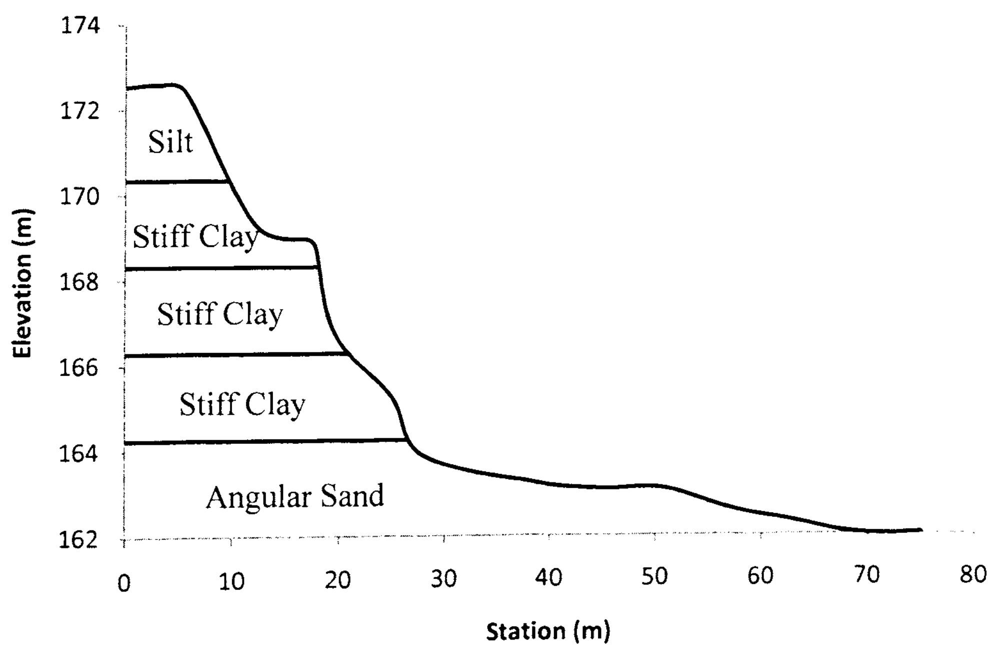

Figure B.2 Cross-Section 3 Geometry and Stratigraphy

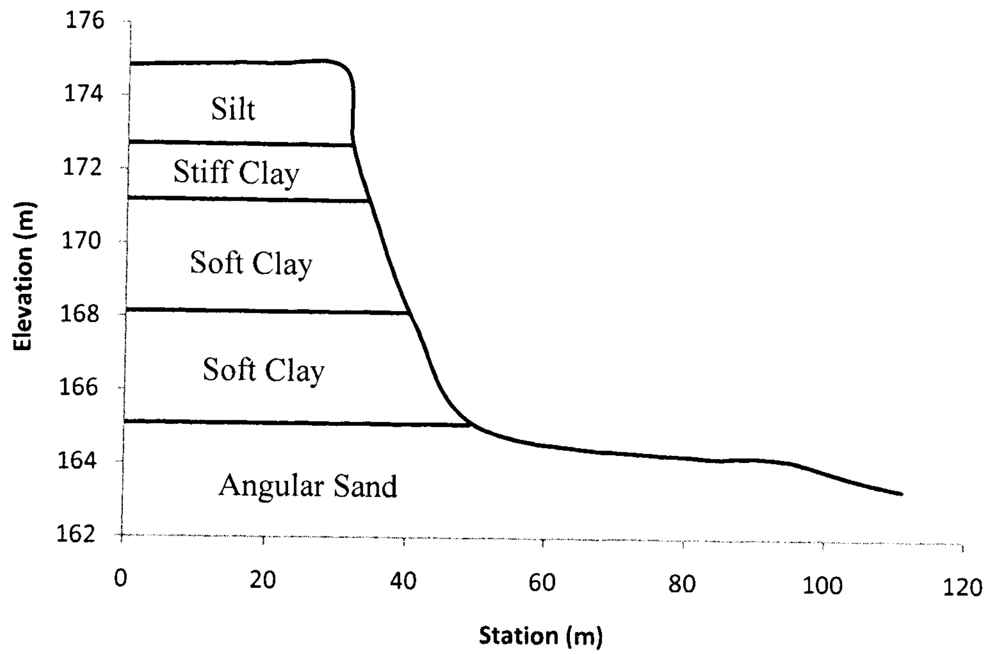

Figure B.3 Cross-Section 4 Geometry and Stratigraphy

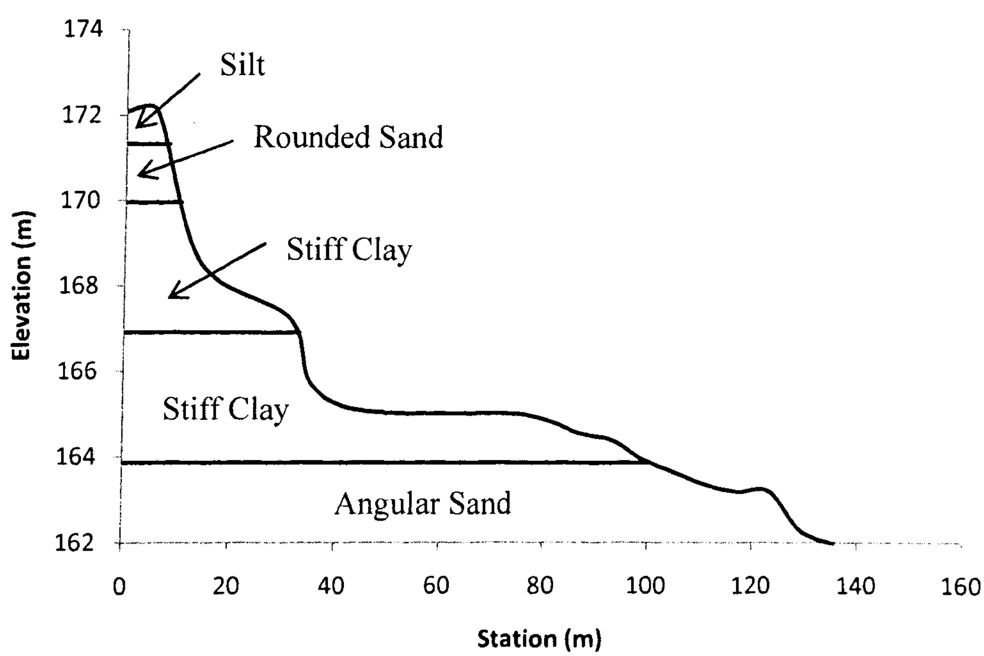

Figure B.4 Cross-Section 5 Geometry and Stratigraphy

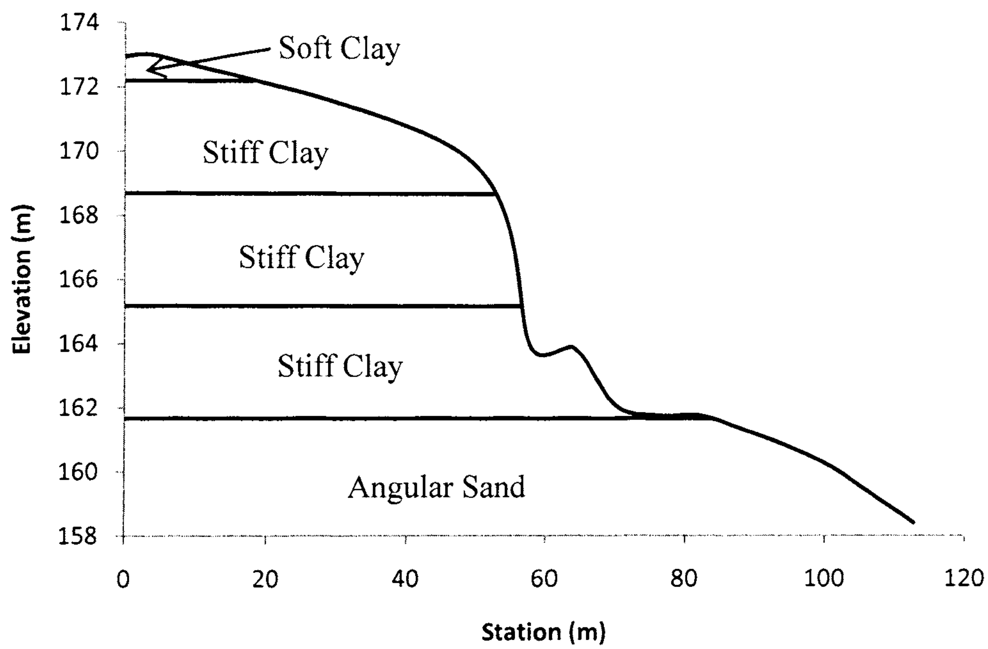

Figure B.5 Cross-Section 6 Geometry and Stratigraphy

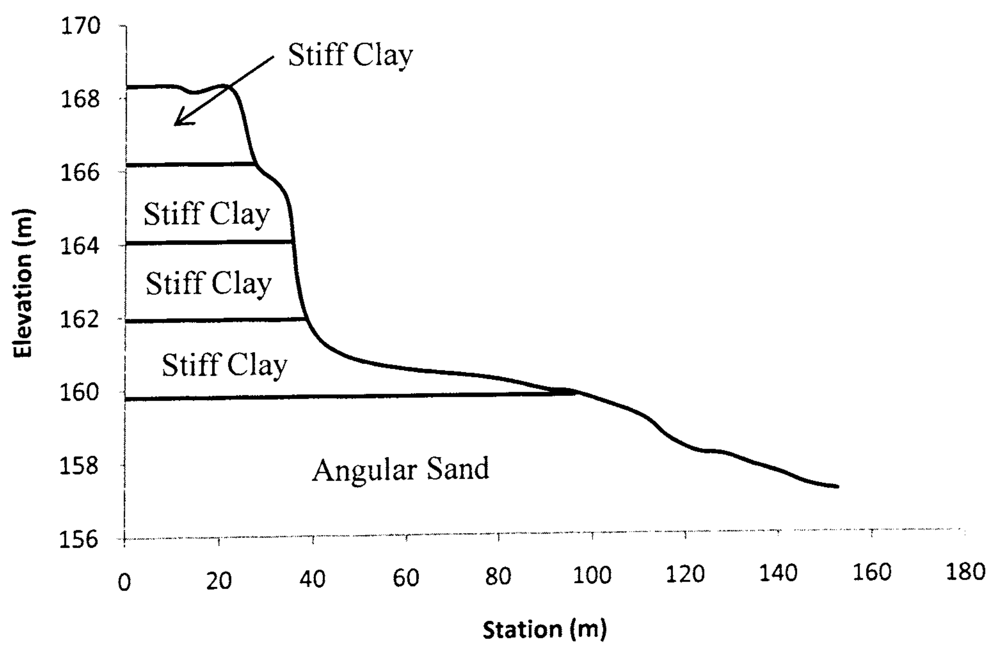

Figure B.6 Cross-Section 8 Geometry and Stratigraphy

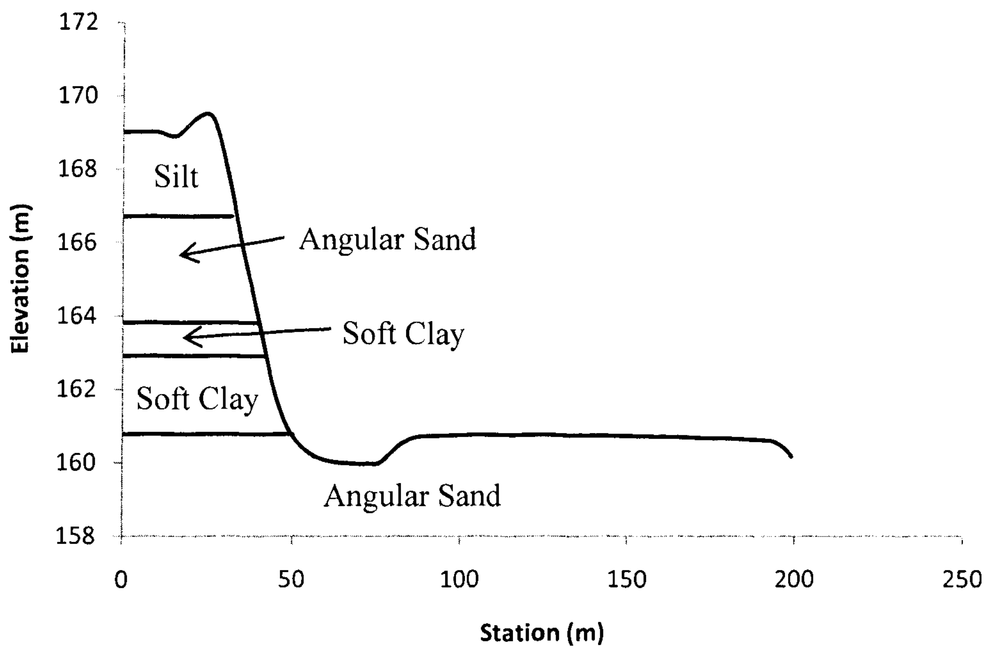

Figure B.7 Cross-Section 9 Geometry and Stratigraphy

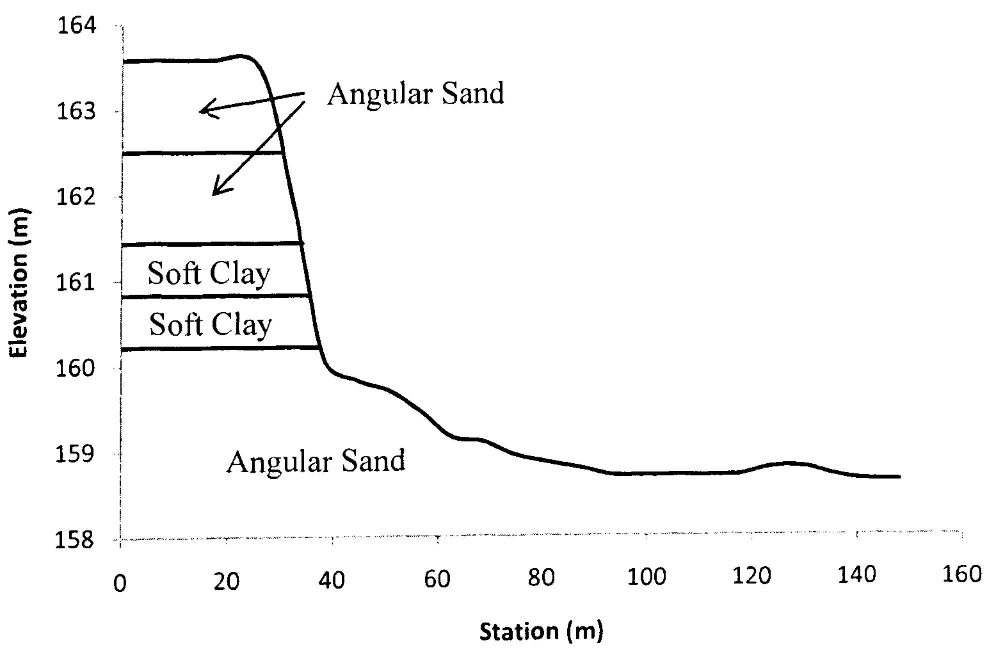

Figure B.8 Cross-Section 10 Geometry and Stratigraphy

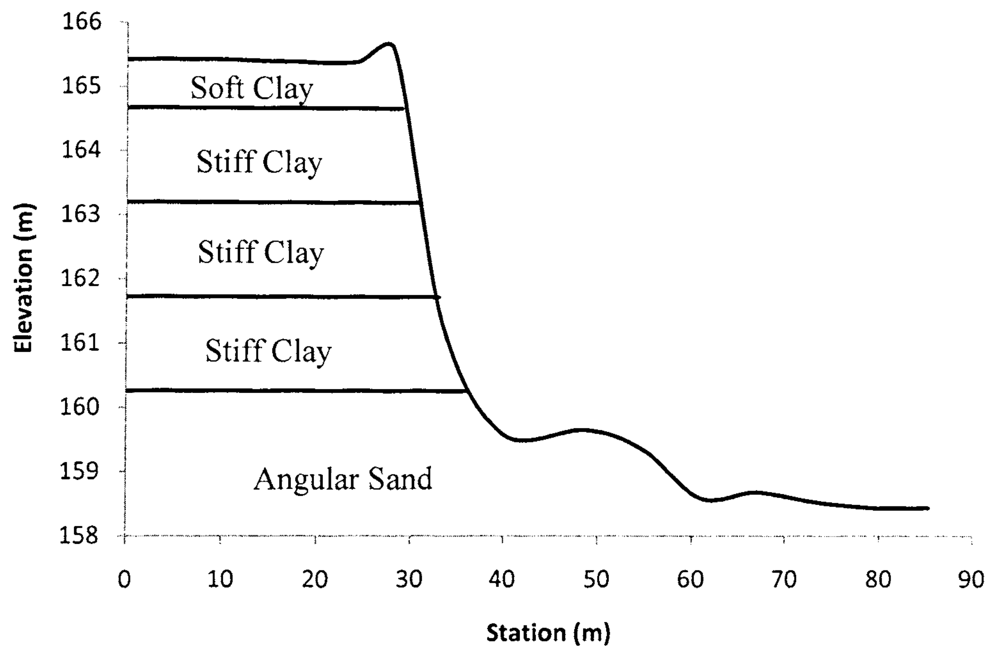

Figure B.9 Cross-Section 11 Geometry and Stratigraphy

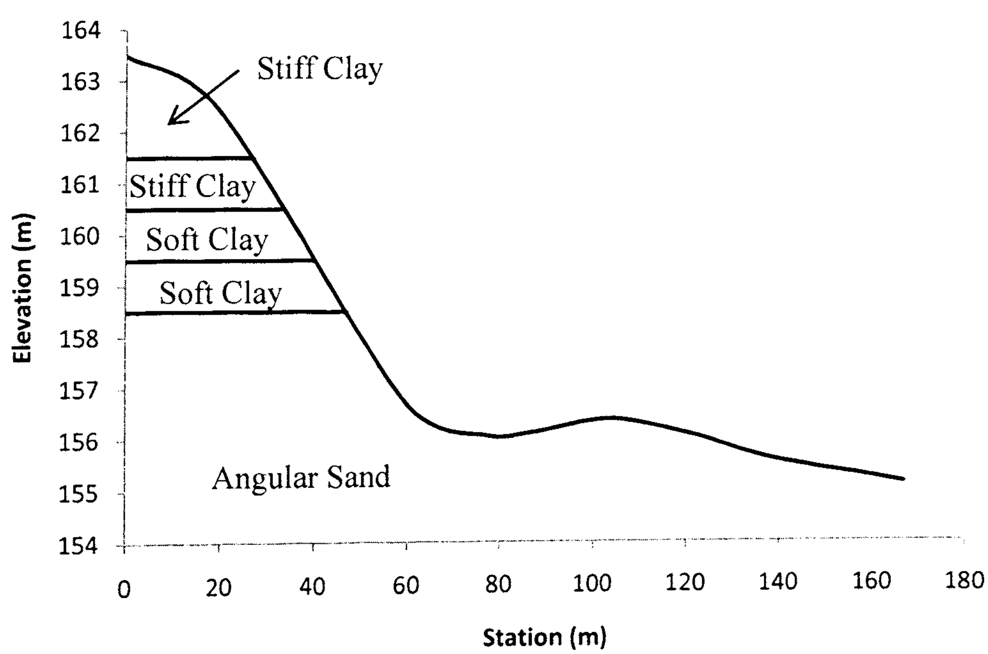

Figure B.10 Cross-Section 14 Geometry and Stratigraphy

APPENDIX C

SENSITIVITY ANALYSIS FACTOR OF SAFETY PLOTS

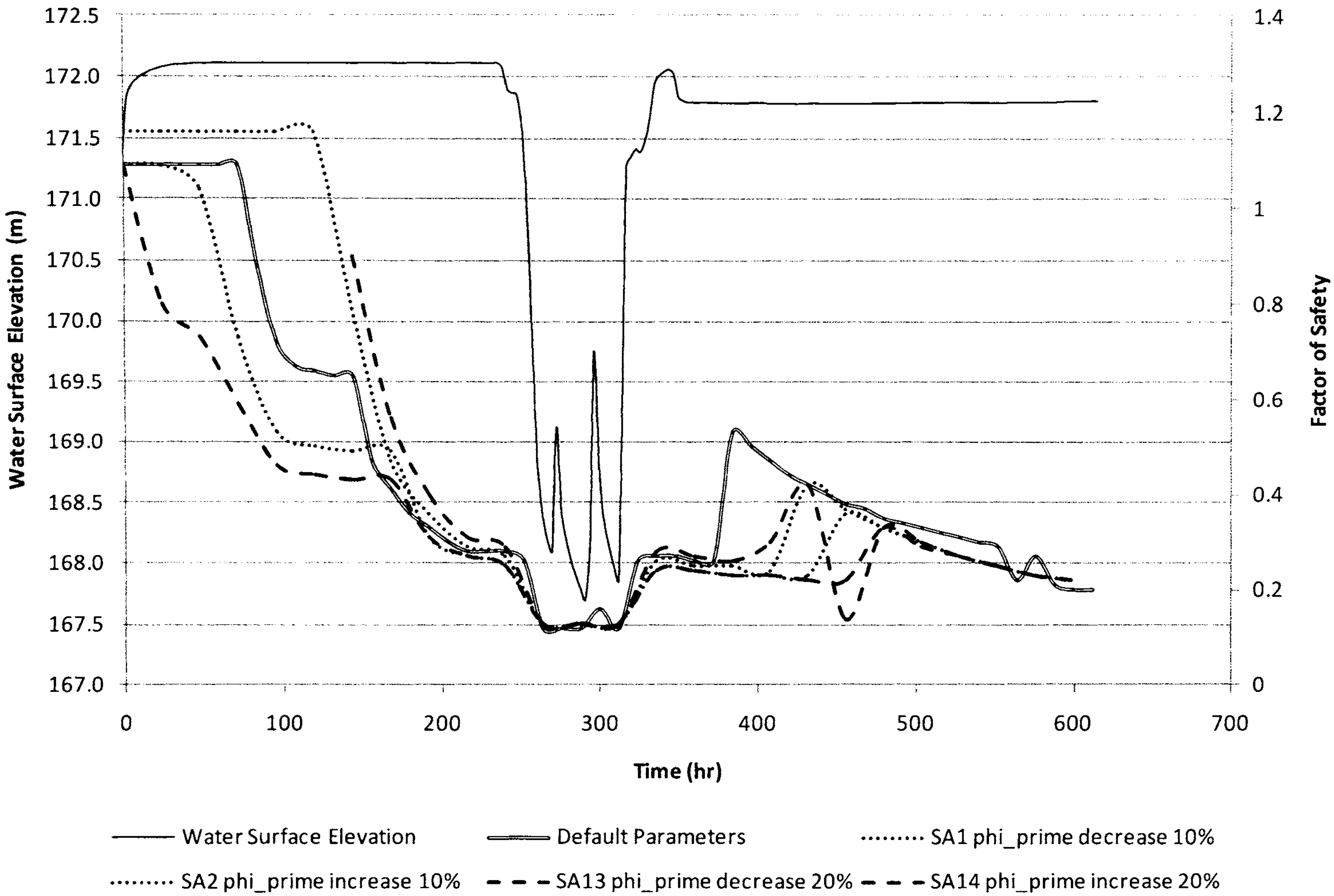

Figure C.1 Results of Sensitivity Analysis: Varying φ'

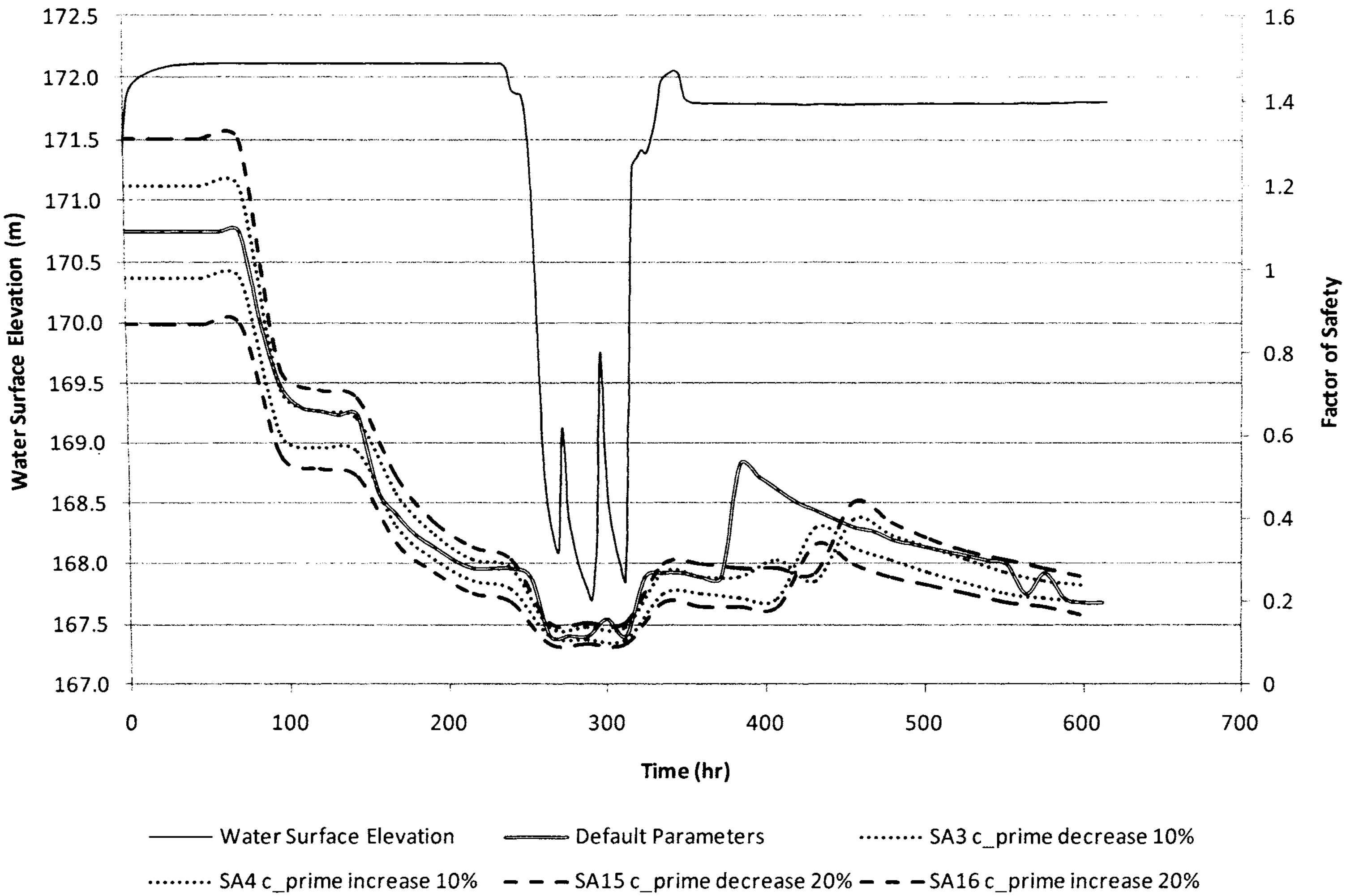

Figure C.2 Results of Sensitivity Analysis: Varying c'

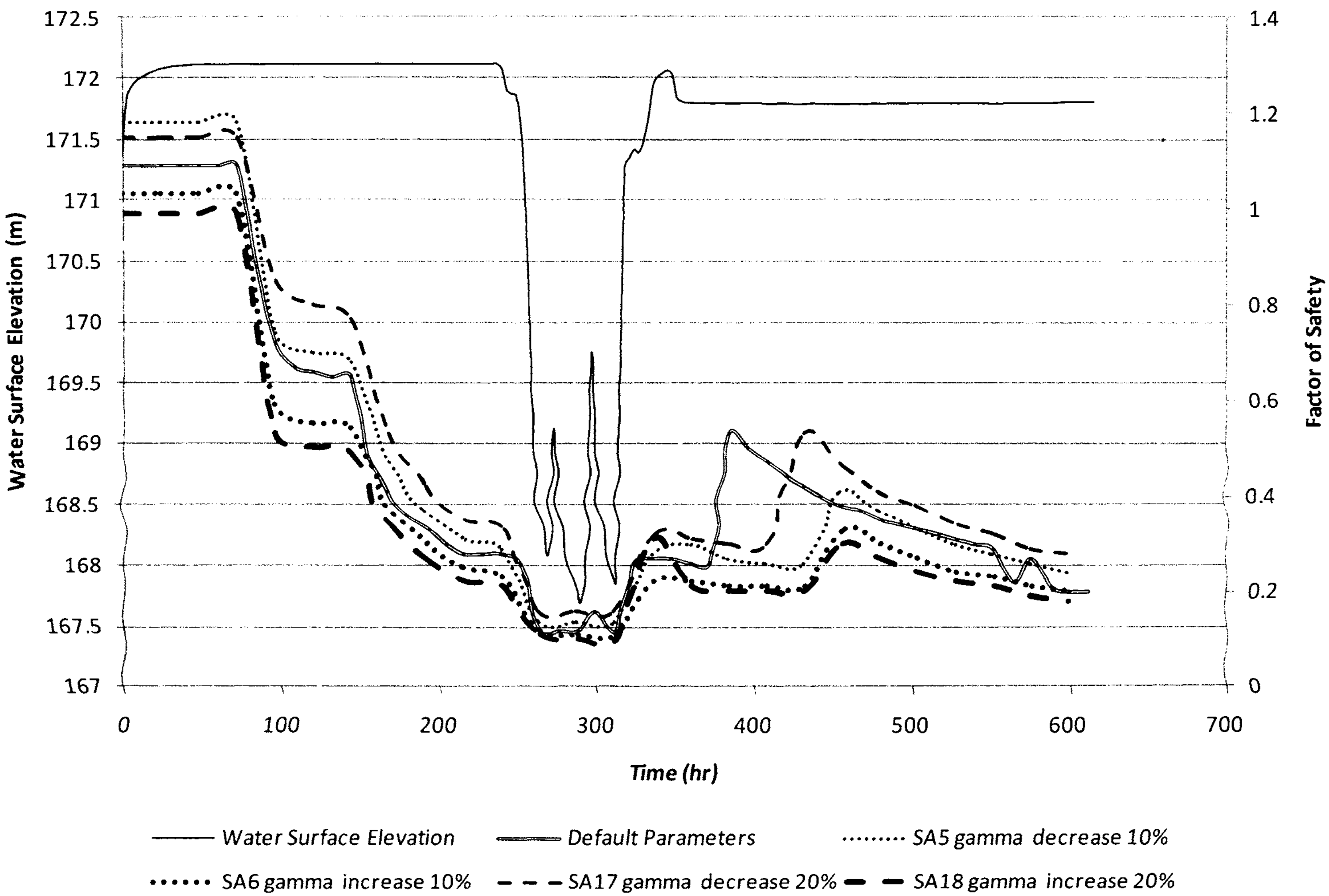

Figure C.3 Results of Sensitivity Analysis: Varying γ

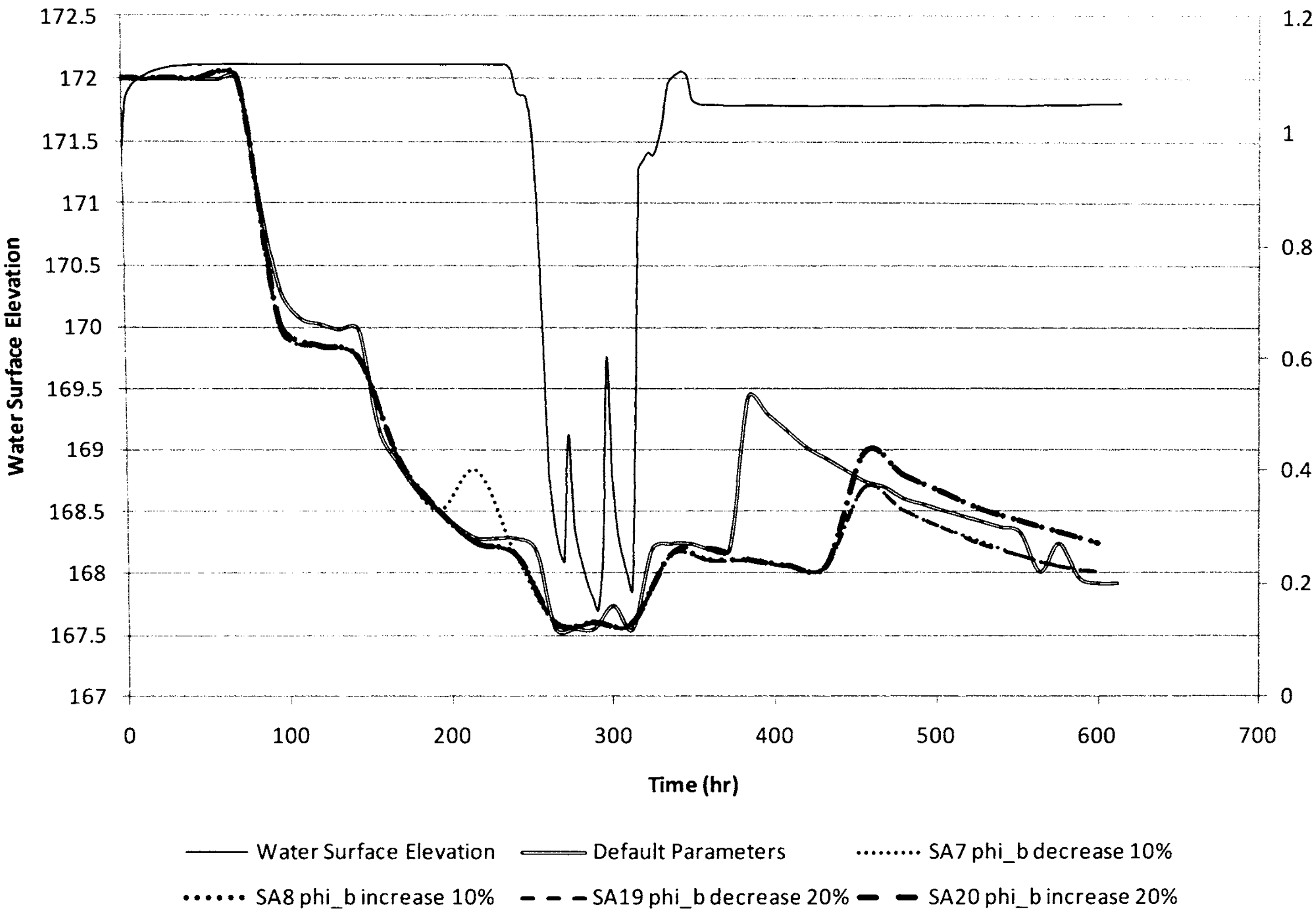

Figure C.4 Results of Sensitivity Analysis: Varying φ_b

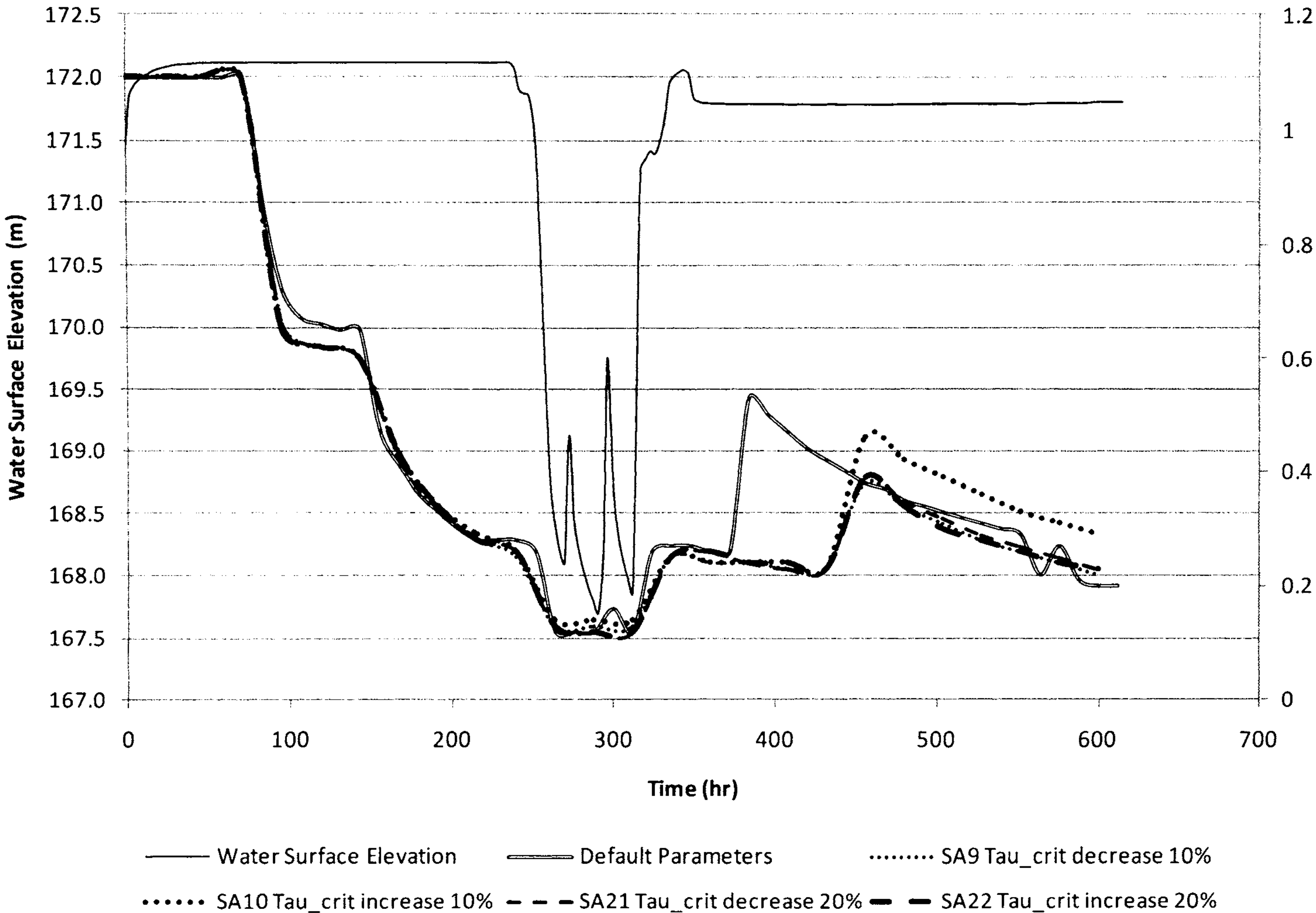

Figure C.5 Results of Sensitivity Analysis: Varying τ_c

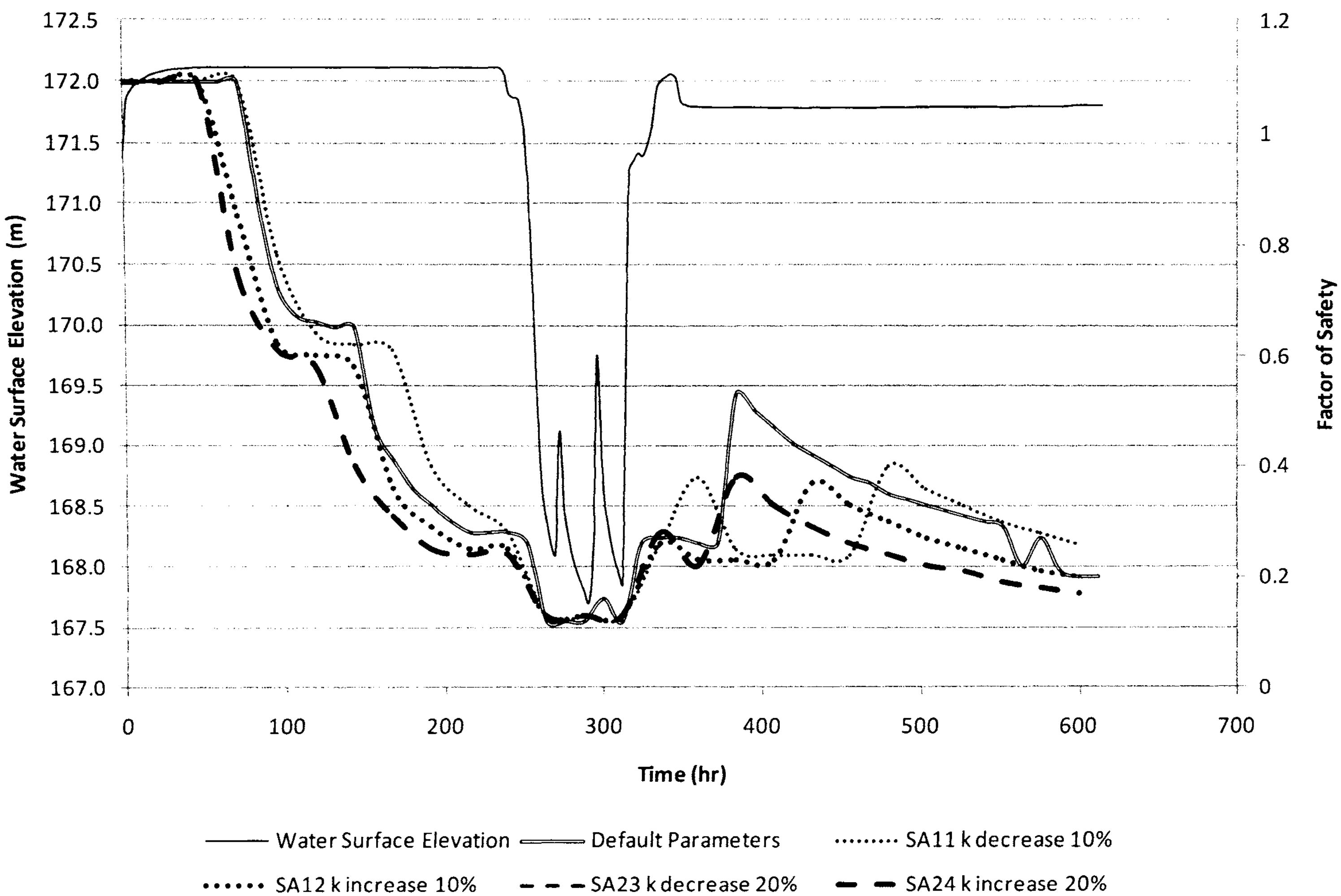

Figure C.6 Results of Sensitivity Analysis: Varying k

APPENDIX D

CRITICAL FACTOR OF SAFETY PLOTS

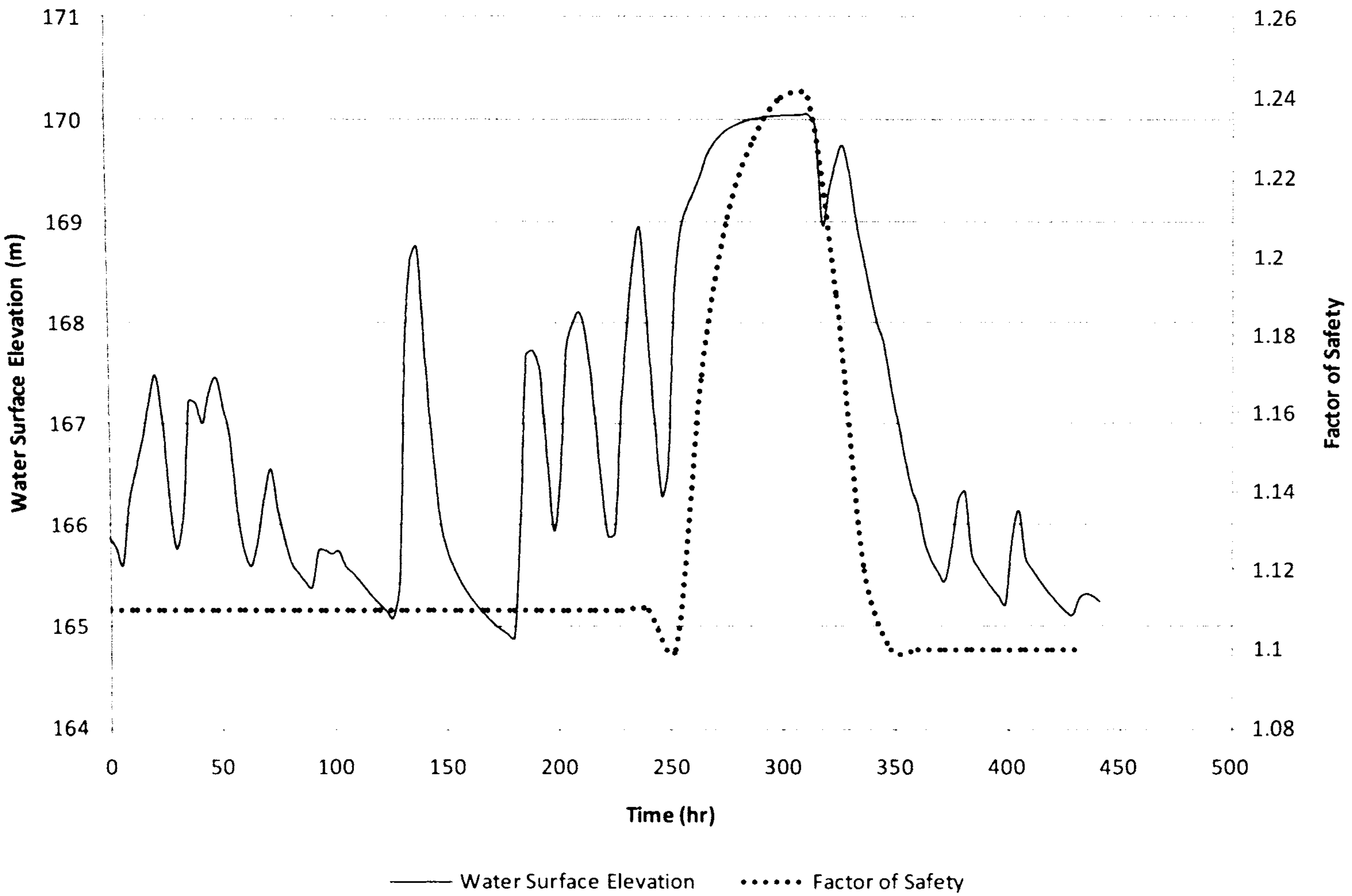

Figure D.1 Hypolimnia Scenario at Cross-Section 4

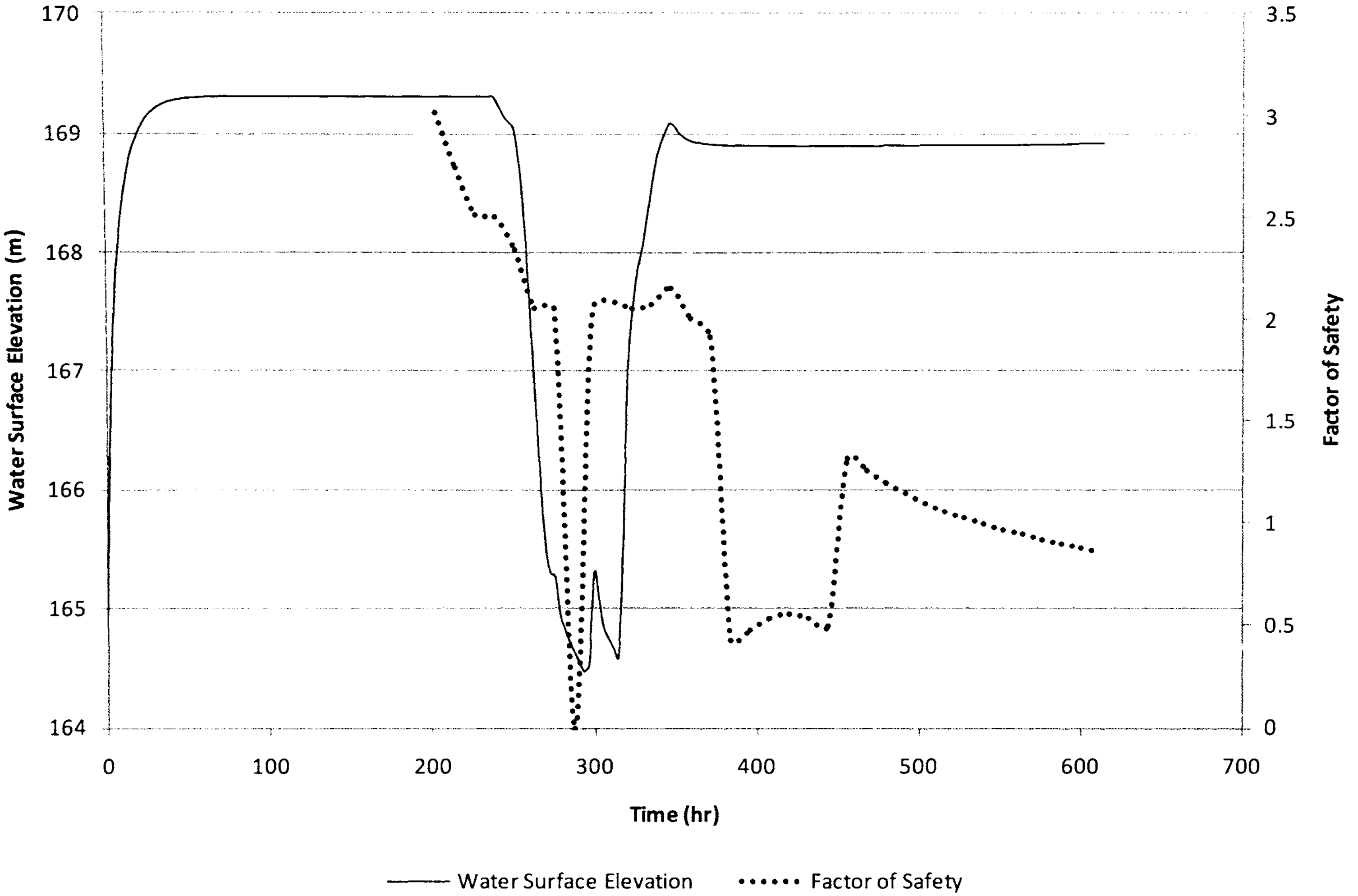

Figure D.2 High Flow Scenario at Cross-Section 5

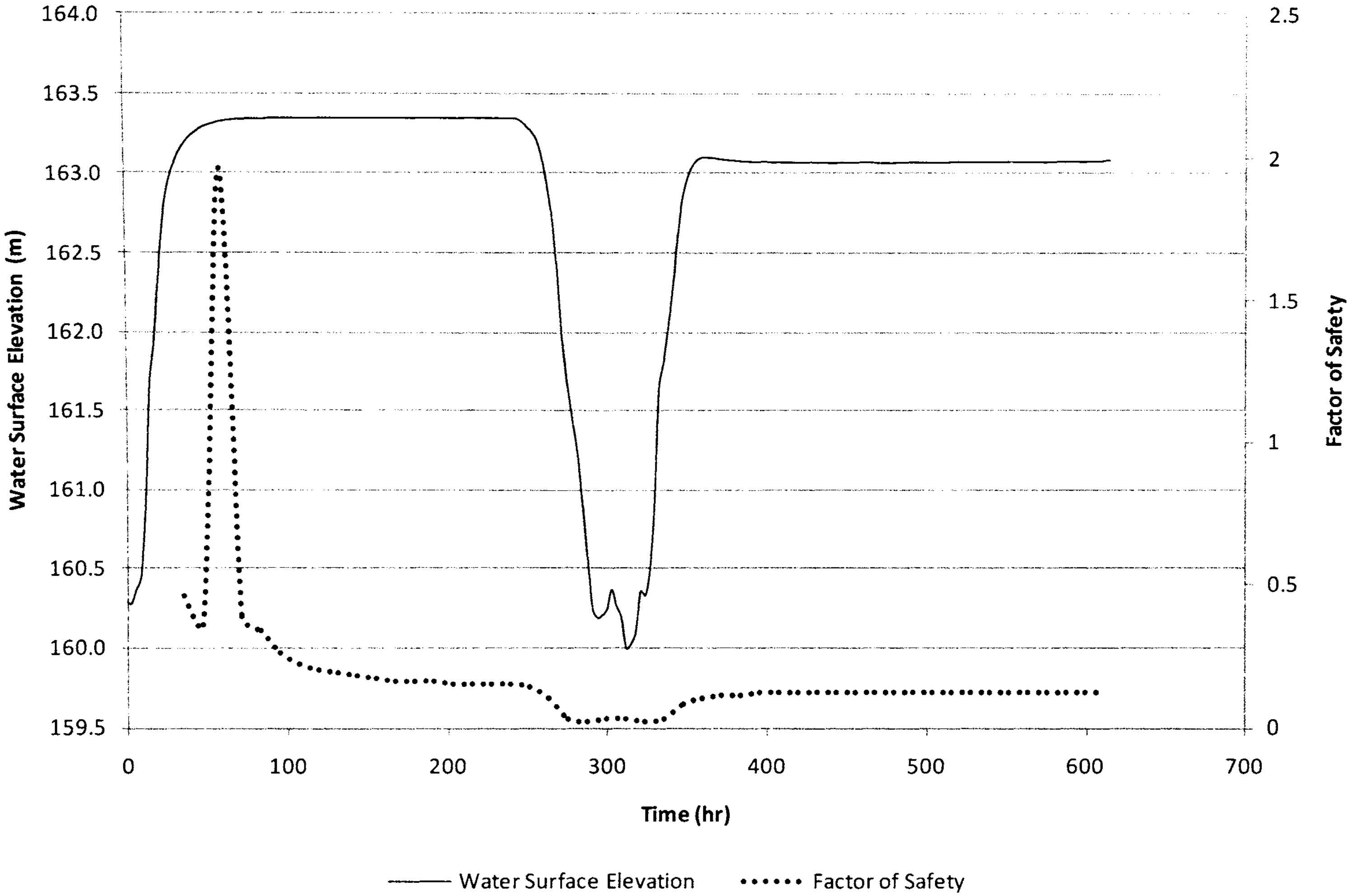

Figure D.3 High Flow Scenario at Cross-Section 10

Figure D.4 Summer Generation Scenario at Cross-Section 11

APPENDIX E

ORIGINAL AND ERODED CROSS-SECTIONS

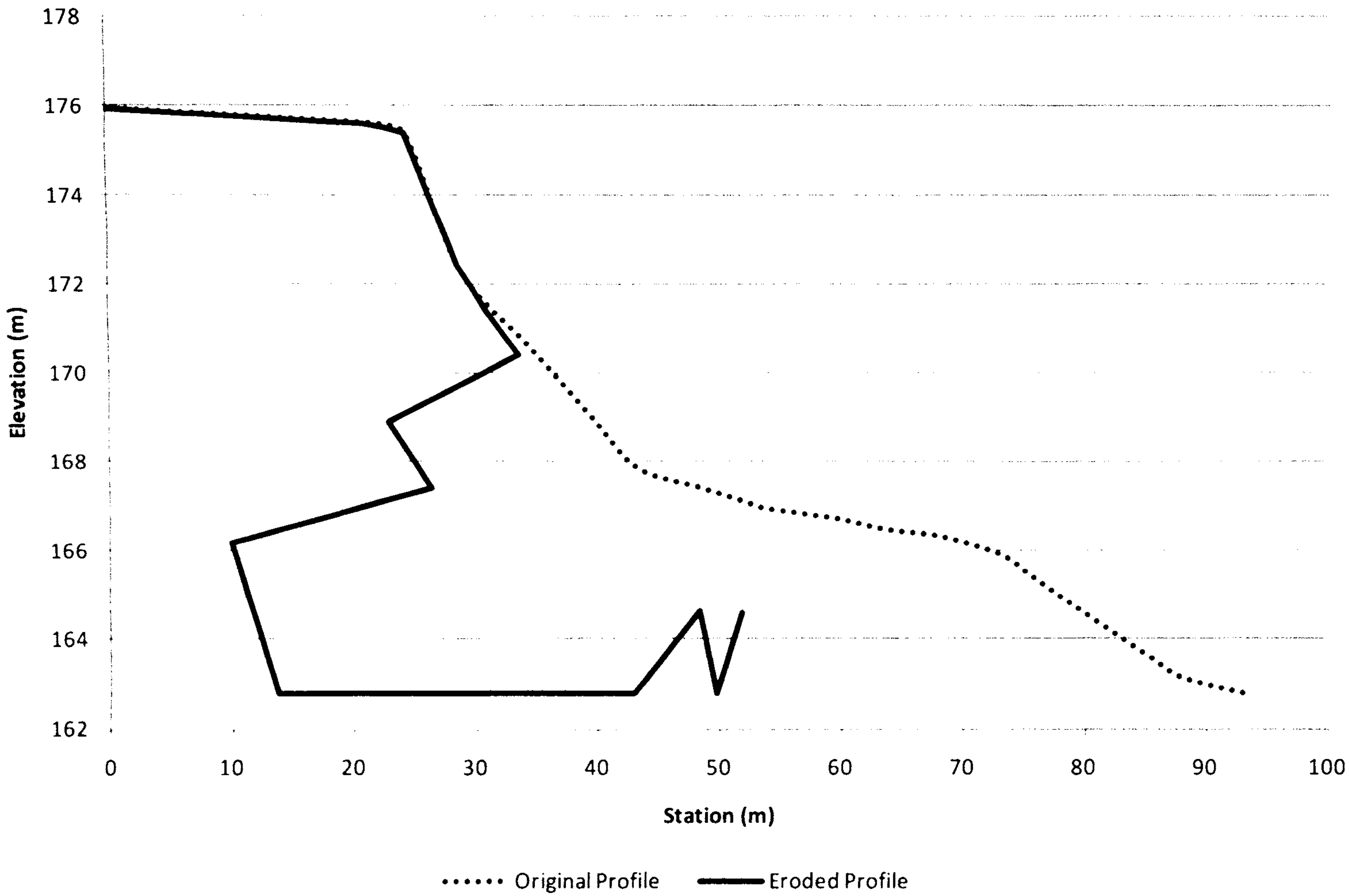

Figure E.1 Erosion Resulting from Summer Generation Scenario at Cross-Section 1

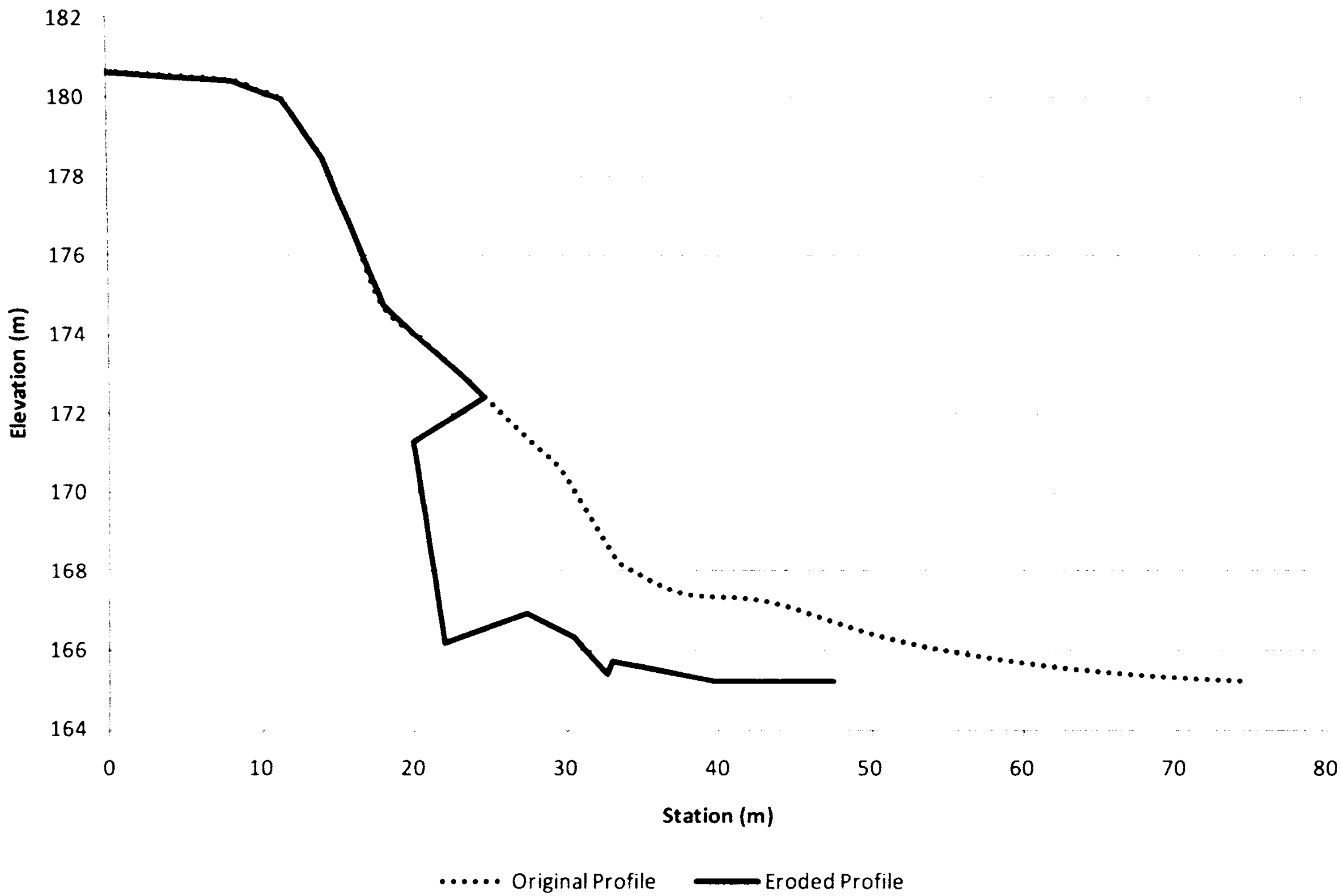

Figure E.2 Erosion Resulting from High Flow Scenario at Cross-Section 2

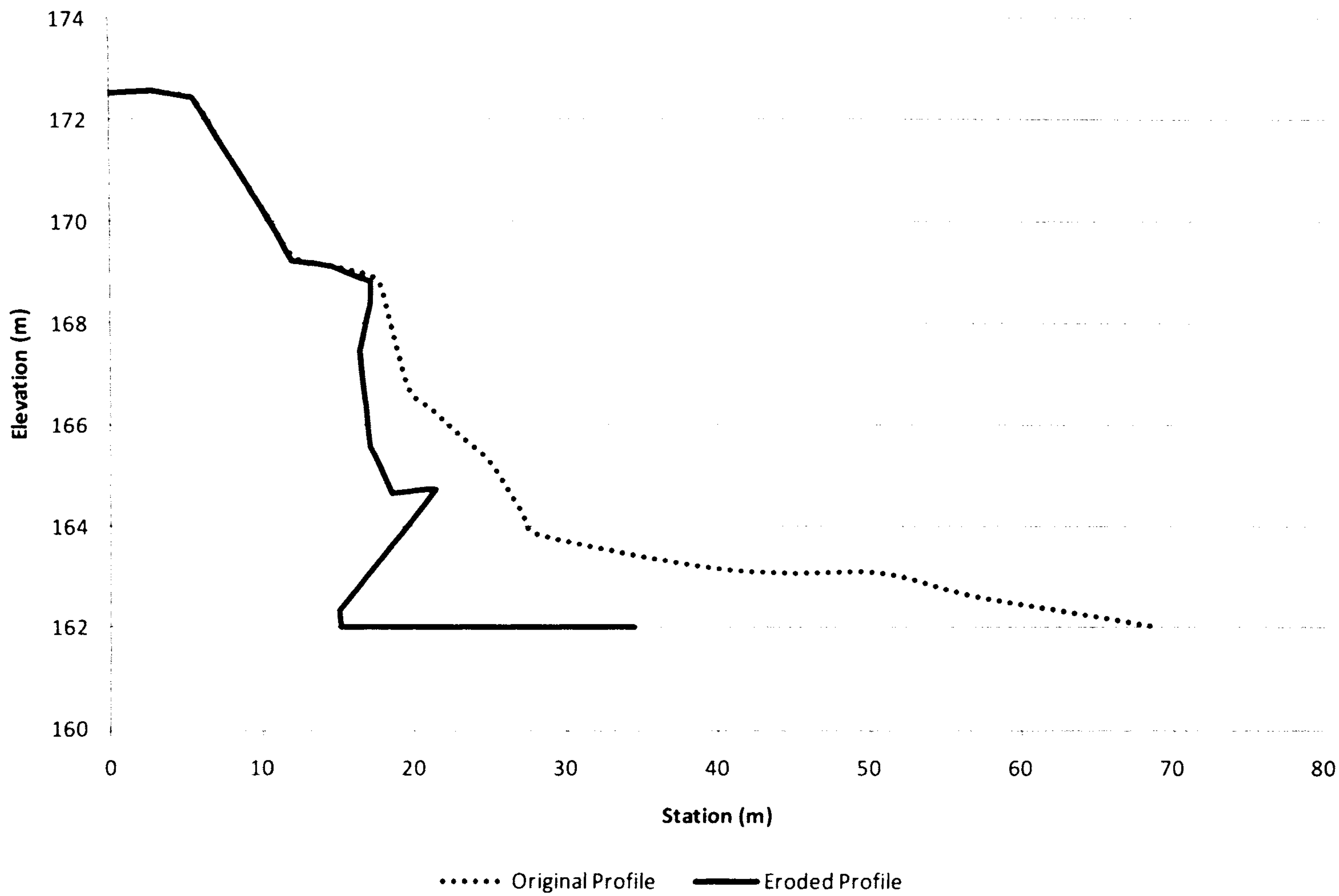

Figure E.3 Erosion Resulting from High Flow Scenario at Cross-Section 3

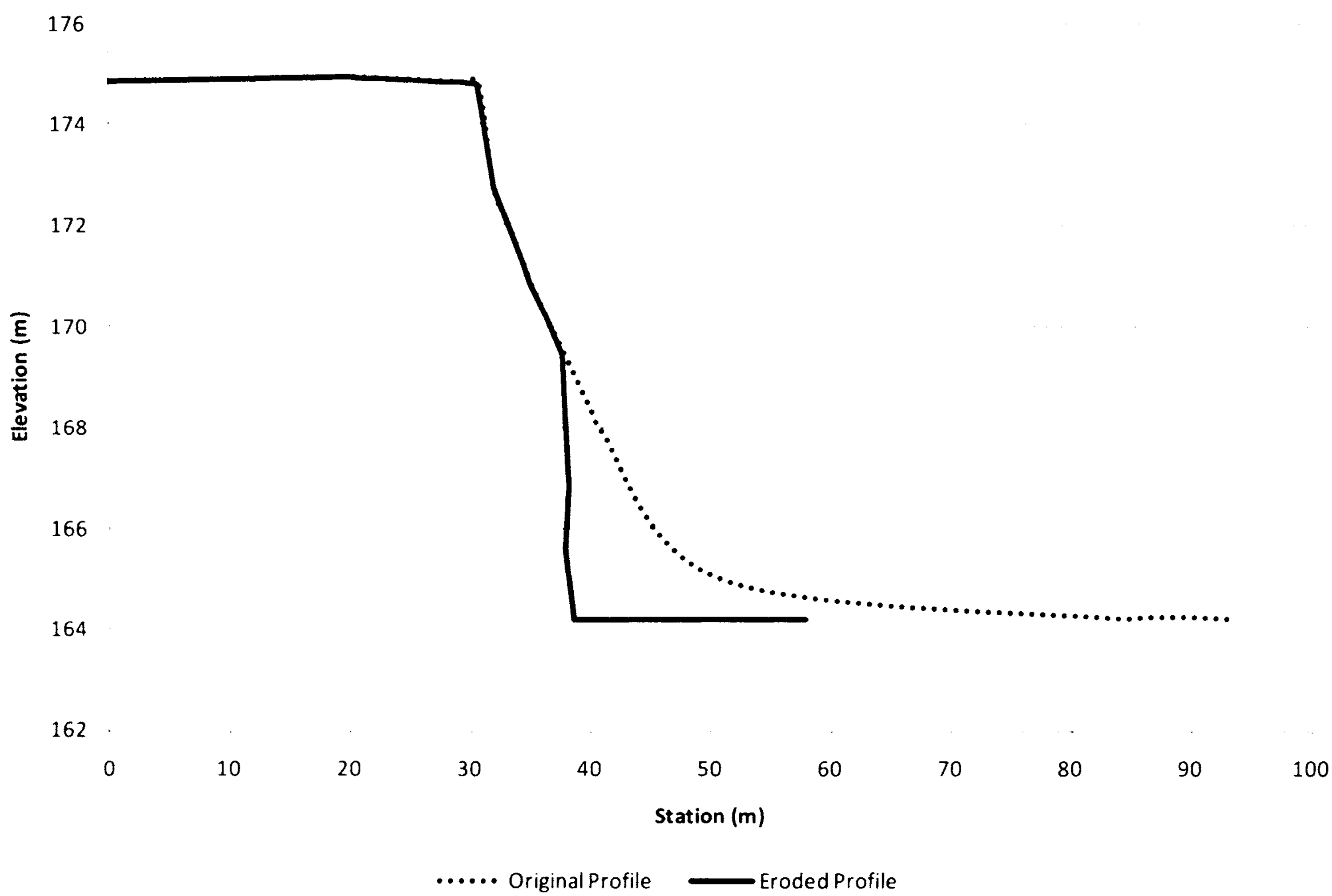

Figure E.4 Erosion Resulting from Hypolimnia Scenario at Cross-Section 4

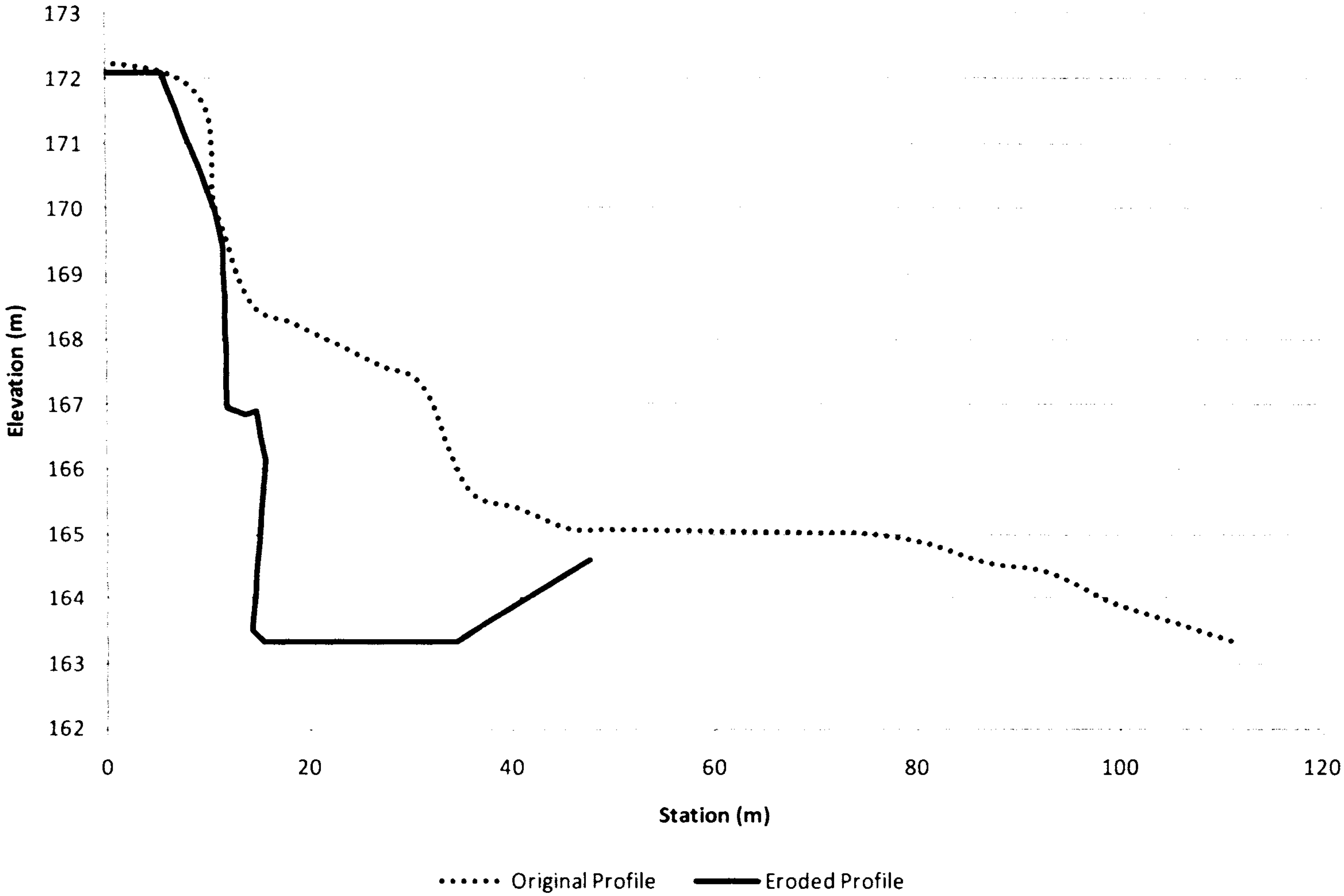

Figure E.5 Erosion Resulting from High Flow Scenario at Cross-Section 5

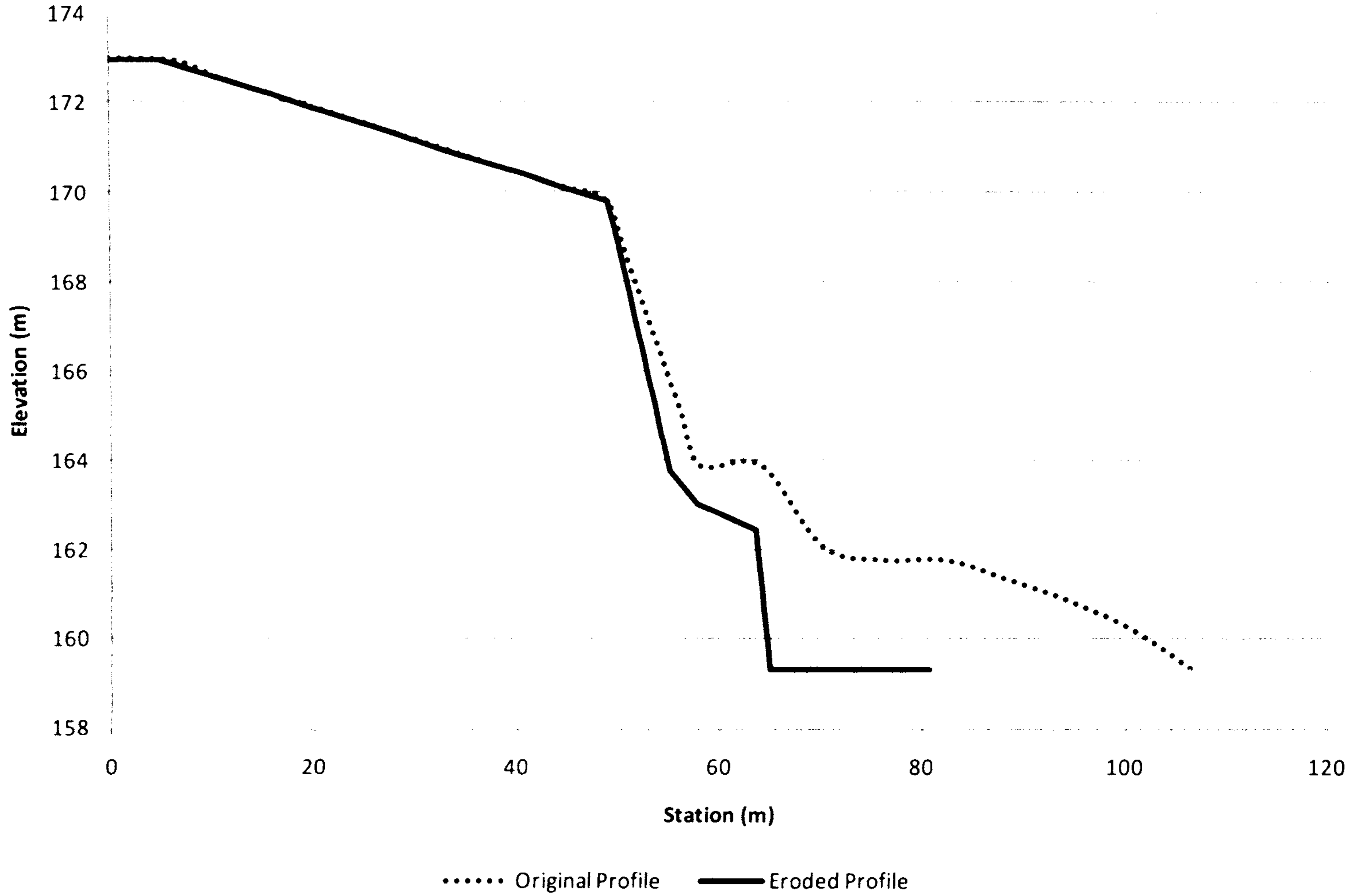

Figure E.6 Erosion Resulting from High Flow Scenario at Cross-Section 6

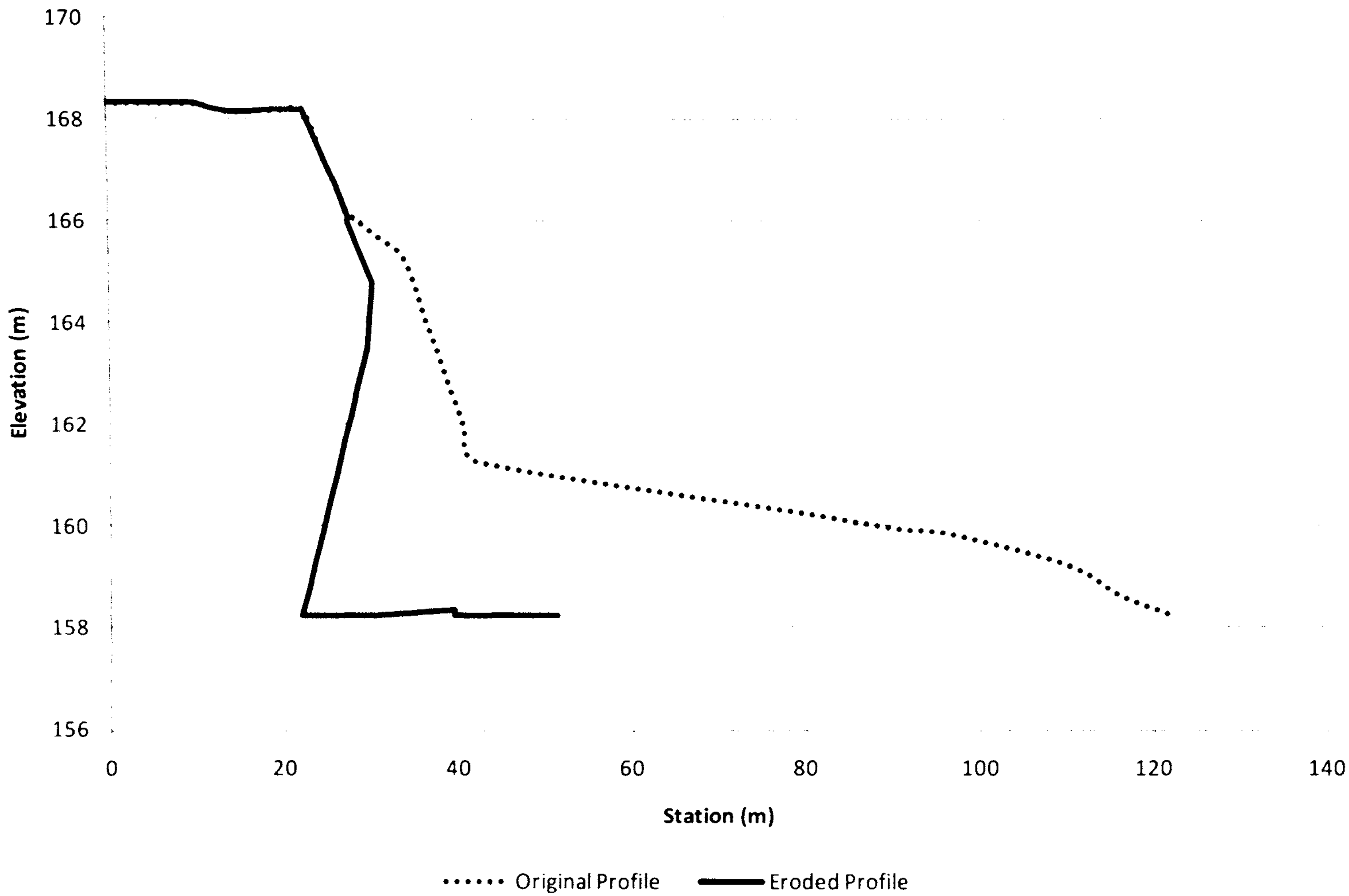

Figure E.7 Erosion Resulting from High Flow Scenario at Cross-Section 8

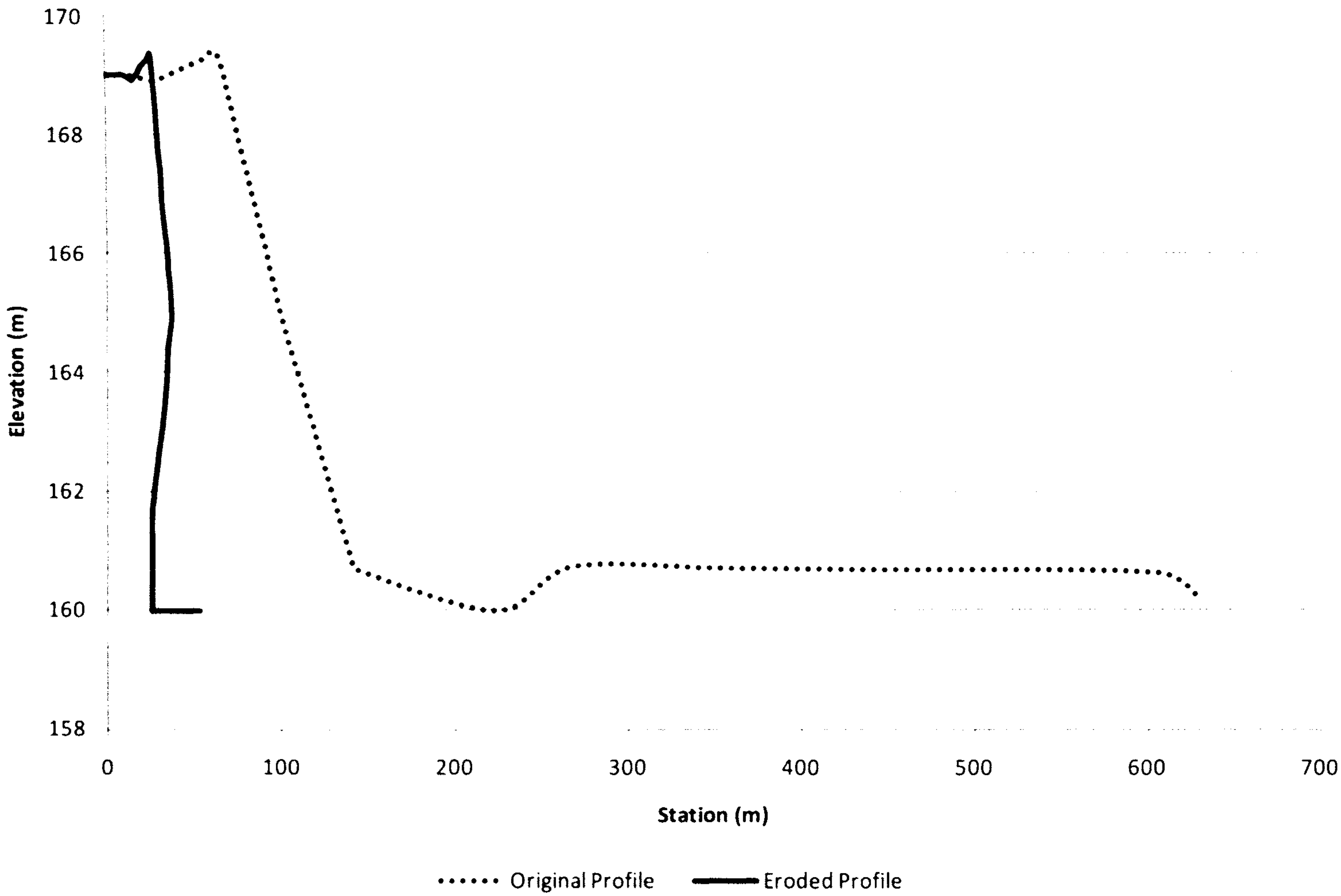

Figure E.8 Erosion Resulting from Summer Generation Scenario at Cross-Section 9

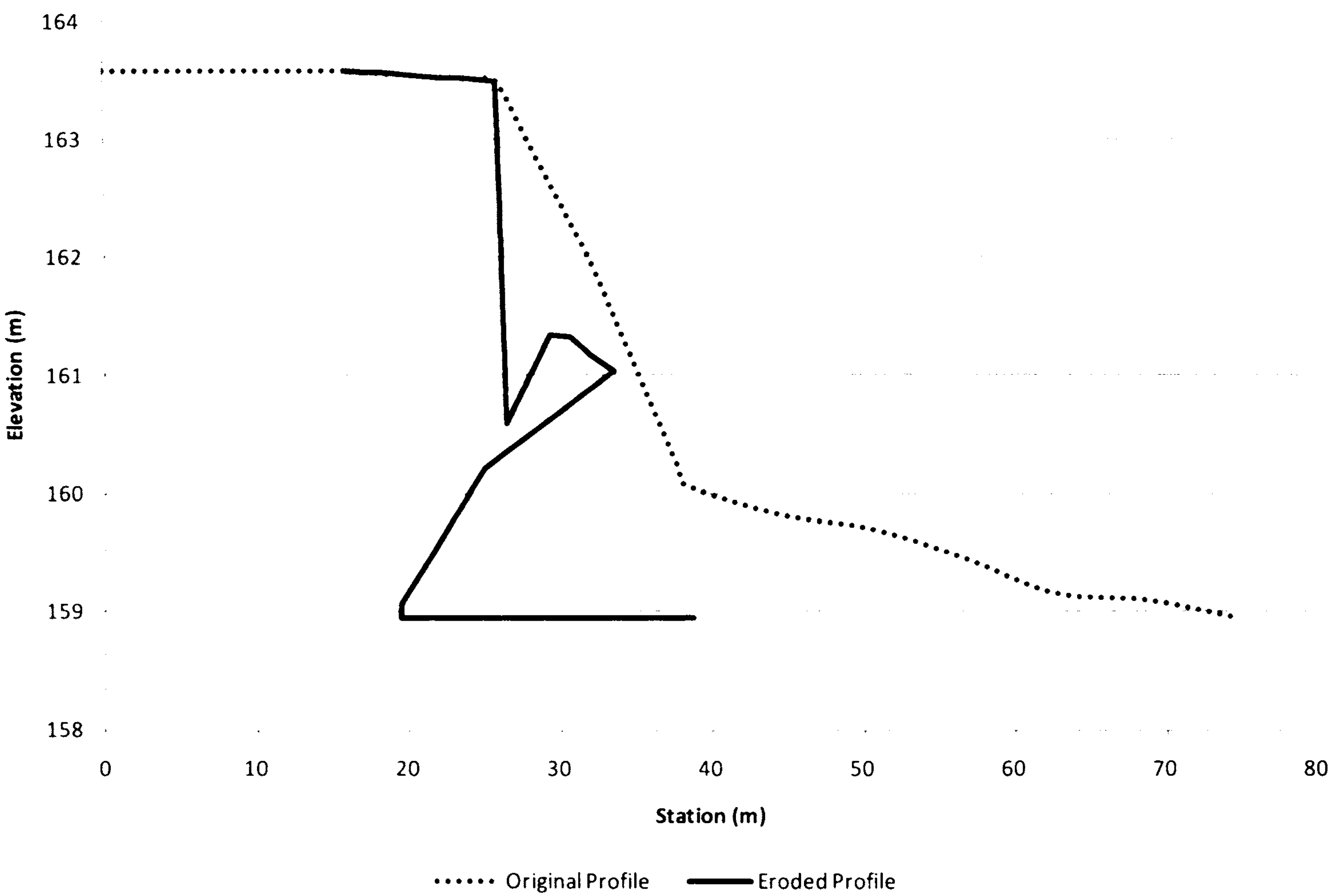

Figure E.9 Erosion Resulting from High Flow Scenario at Cross-Section 10

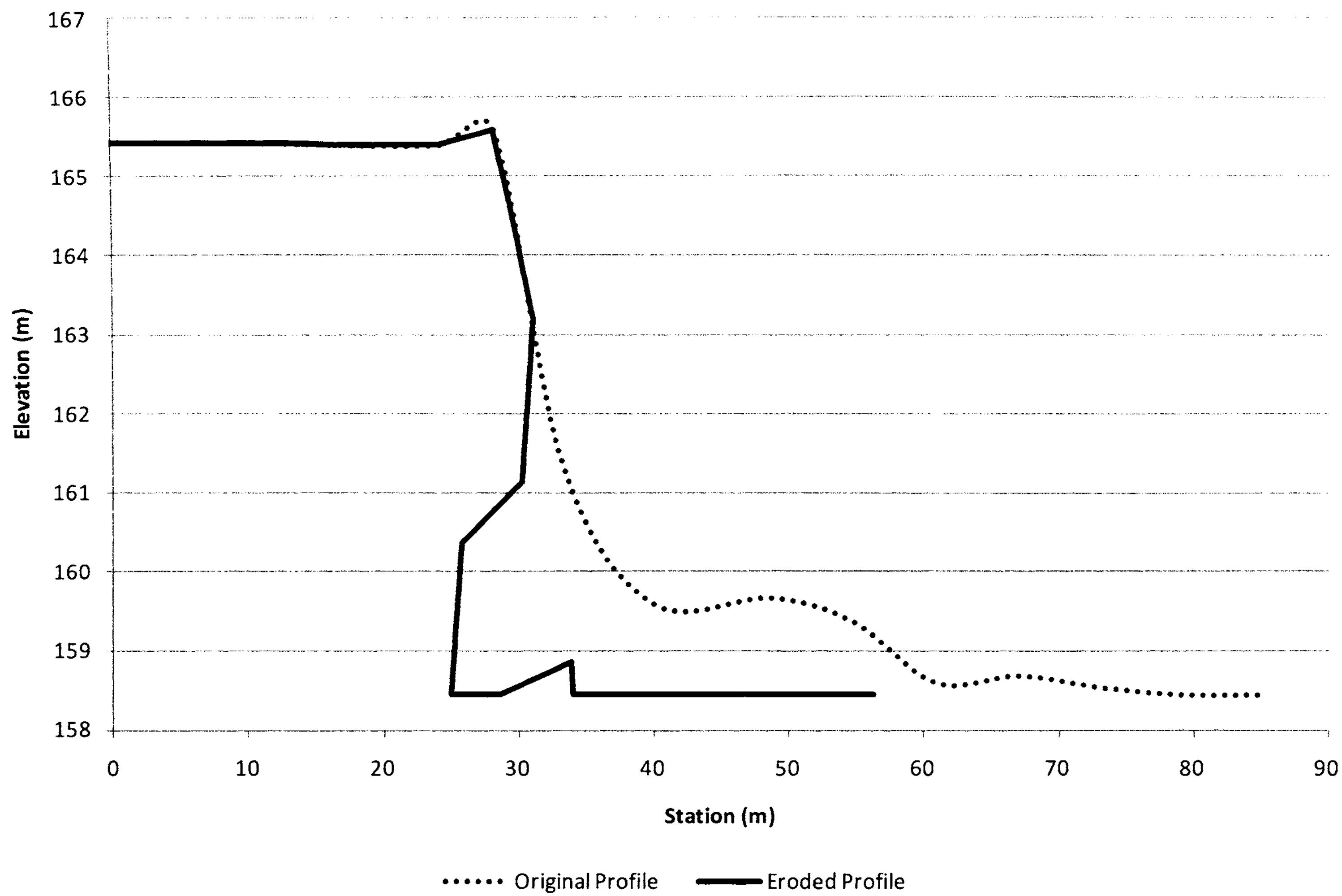

Figure E.10 Erosion Resulting from High Flow Scenario at Cross-Section 11

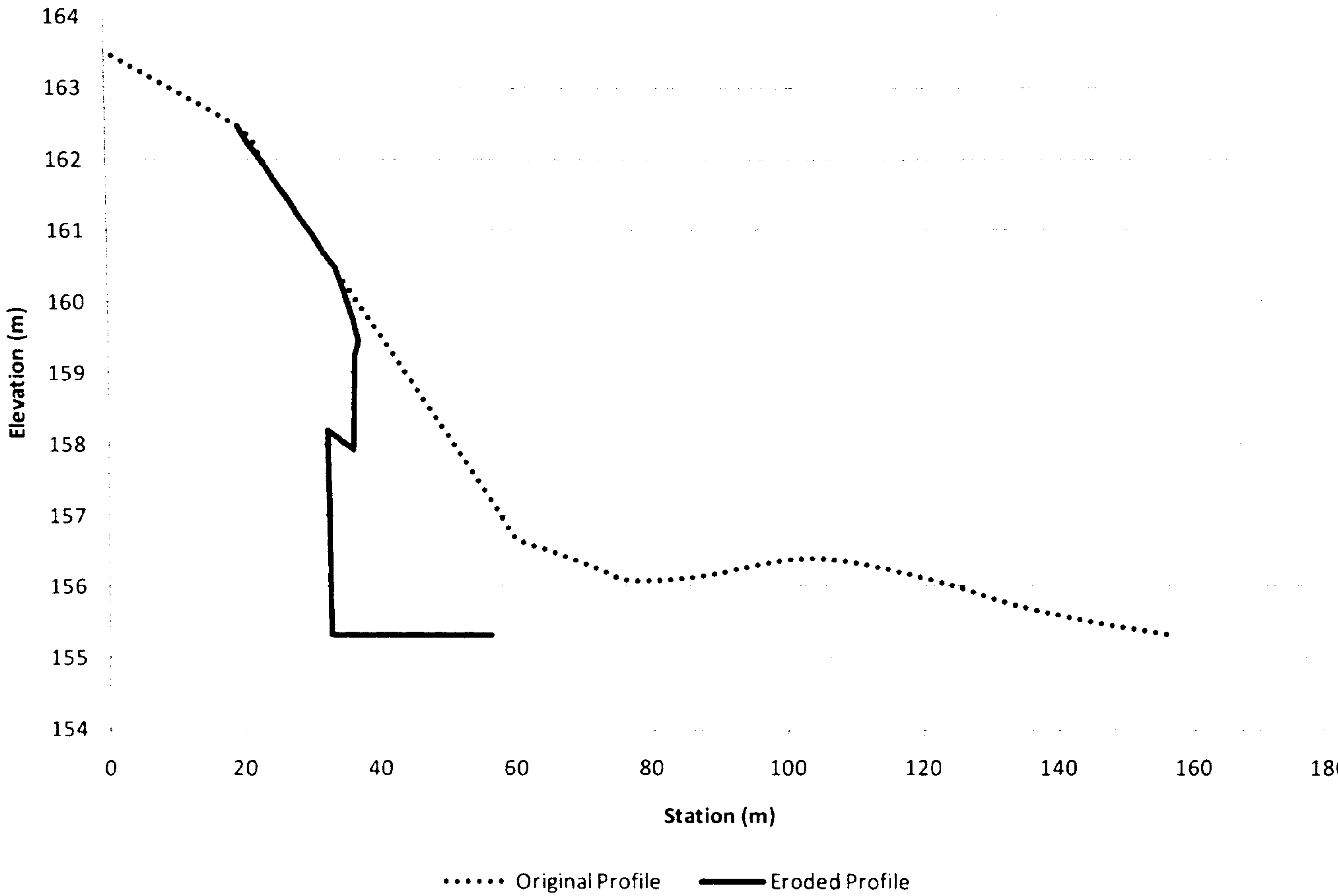

Figure E.11 Erosion Resulting from High Flow Scenario at Cross-Section 14

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